TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. LIV.
JUNE, 1905.

Edited by the Secretary, under the direction of the Committee on Publications.

NEW YORK:
PUBLISHED BY THE SOCIETY.

1905.

CONTENTS.

PAPERS.

MEMOIRS OF DECEASED MEMBERS.

PLATES.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 988.

THE INSTALLATION OF A PNEUMATIC PUMPING PLANT.*

By Arthur H. Diamant, Jun. Am. Soc. C. E.

With Discussion by Messrs. Elmo G. Harris and Edward Wegmann.

Before proceeding with the description of the pumping plant, which is to be used in case of emergency only, the writer deems it advisable to give a brief statement as to the necessity for its installation.

As is generally known, the City of New York receives its water supply through the New Croton Aqueduct, which begins at the inlet gate-house, near the Old Croton Dam, Croton Lake, and, after reaching Shaft No. 24, on the Bronx side of Washington Bridge and near it, passes under the Harlem River to Shaft No. 25 and thence to the terminal gate-house at One Hundred and Thirty-fifth Street, near Amsterdam Avenue.

Provision has been made for emptying the aqueduct whenever necessary. The inlet gates at Croton Lake can be closed, thus preventing water from entering the aqueduct. To empty that portion between Croton Lake and Washington Bridge, there are blow-off gates

* Presented at the meeting of September 7th, 1904.

at Shaft No. 9, Pocantico; at Shaft No. 14, Ardsley; at Shaft No. 18, South Yonkers; and at Shaft No. 24, near Washington Bridge. There are also blow-off pipes on the stretch between Shaft No. 25 and the terminal gate-house, so that the aqueduct can be made to empty itself, with the exception of that portion constituting the Harlem River crossing or siphon, Fig. 1. This siphon is emptied by Shaft No. 25, the pump-shaft.

Shaft No. 25 (see Fig. 2) is really a double shaft, the northerly one being the aqueduct-shaft, and the southerly one the pump-shaft.

The aqueduct-shaft is 12.25 ft. in diameter, and, at a point about 10 ft. above high water, the aqueduct continues on its way to the terminal gate-house. The pump-shaft, also 12.25 ft. in diameter, is completely lined with iron, and contains a sump extending 21.75 ft. below the bottom of the siphon tunnel. An opening, 1 ft. 8 in. by 2 ft. 6 in., and 3 ft. below the invert of the tunnel, regulated by a gate, admits the water into the pump-shaft. This gate, being 417 ft. below the top of the shaft, is of composition metal, moving in solid composition grooves, and is designed so that no obstructions can accumulate in the frame. It is raised by a square stem, 3.5 by 3.5 in., guided every 12 ft., and contained in a 3-ft. pipe built in the masonry. This pipe also contains a ladder reaching from the top (Elevation 84.5) to the bottom (Elevation—312.75). A plan of the connection is shown in the "Section through *A B*," Plate I. As each shaft is under the hydraulic grade, it can be closed by a double set of manholes with covers. For the purpose of blowing off the water, each shaft is connected with a 48-in. cast-iron pipe, with two gates, and discharging into the river.

Over the pump-shaft was erected a bucket-hoist, composed of two alternating buckets. each of 1 390 gal. capacity. These were raised and lowered by a horizontal steam engine capable of emptying each in 0.5 min. Fig. 3, prepared by Mr. F. S. Cook, Engineer in Charge of the Draughting Bureau of the Aqueduct Commissioners, shows the volumes of water to be lifted in emptying the siphon. With this plant, it would have taken from 15 to 18 hours to accomplish this task, provided the engines could have continued at the aforesaid rate. As shutting down the aqueduct would entail serious inconveniences, the present water consumption being about 296 000 000 gal. per day, the Aqueduct Commissioners deemed it necessary to install a pumping

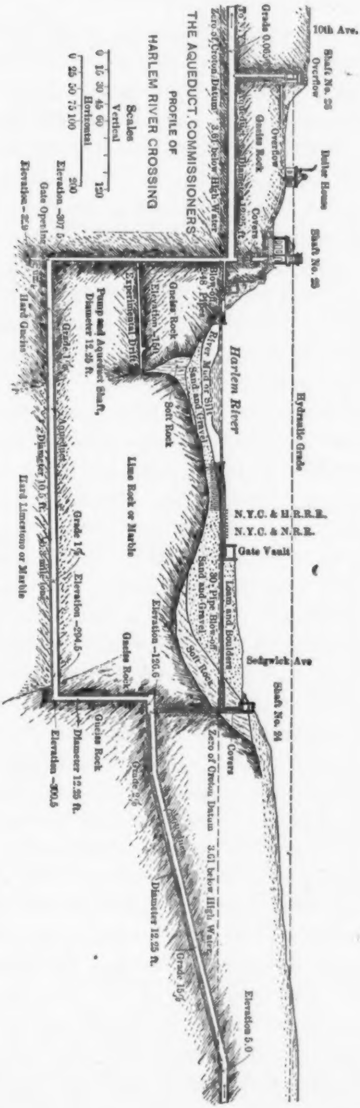

THE AQUEDUCT COMMISSIONERS
PROFILE OF
HARLEM RIVER CROSSING
Scales
Vertical
Horizontal
10th Ave.
Shaft No. 23
Overflow
Boiler House
Shaft No. 25
Hydraulic Grade
Harlem River
River Mud or Silt
Sand and Gravel
Gneiss Rock
Soft Rock
Experimental Drift
Pump and Aqueduct Shaft, Diameter 12.25 ft.
Lime Rock or Marble
Hard Gneiss
Hard Limestone or Marble
N.Y.C. & H.R.R.R.
N.Y.C. & N.R.R.
Gate Vault
Loam and Boulders
Elevation -126.6
Elevation -294.5
Sedgwick Ave
Shaft No. 24
Covers
Zero of Croton Datum 3.61 below High Water
Elevation -300.5
Diameter 12.25 ft.
Elevation 5.0

FIG. 1.

plant which could empty the siphon in 12 hours or less, as every hour gained would be of material advantage. Accordingly, bids were received for such a plant, and the contract was awarded to the Pneumatic Engineering Company, who proceeded to install the Harris System of pneumatic pump. The contract specified that 2 500 000 gal. be raised 337 ft. in 12 hours, with a bonus of $5 000 for every hour less than 12 hours, and a penalty for each hour longer.

This system, briefly described, consists of a 27 by 48-in. Comstock compressor, twin-connected; the steam engine, 24 by 48-in., being the improved horizontal type, with Corliss valves. Free air, being compressed, passes through coolers, through a switch apparatus, down pipes into four water tanks working in pairs at the bottom of the pump-shaft. An auxiliary compressor supplies the necessary air for running the plant with the greatest efficiency. (See Plate III.) The system is described more fully in the latter part of this paper.

The installation was attended with peculiar difficulties. Leaks have developed in the pump-shaft, since its construction, keeping it full of water up to the blow-off pipe. Weir measurements taken in this pipe show a leakage of about 200 gal. per minute. As there was no way of emptying the shaft and keeping it empty, all work had to be done on an erecting platform built near the blow-off pipe. A 5-in. ejector kept the water about 15 ft. below the platform.

As before stated, the shaft is below the hydraulic grade. If the gate between the aqueduct-shaft and the pump-shaft were to be opened, and the blow-off gates closed, it would be necessary to put on the covers of the manholes in the diaphragms. For this reason, the air-pipes leading down to the four water tanks (see Plate I) could not pass through the manhole openings, but had to pass through four holes bored through the brickwork and iron lining of the two diaphragms. As seen in the drawing, these diaphragms are each about 9 ft. thick, with a space of 6.7 ft. between them. The Rand Drill Company's Davis Calyx drill was used in making the four holes, each 9 in. in diameter, steel shot being used for the cutting surface. Cores, from 3 to 4 ft. long were taken out, showing the efficiency of drills of this style.

For the purpose of lining these holes, and making them continuous between diaphragms, a 6.75-in. cast-iron pipe, with a flange at one end, was placed in each hole of the upper diaphragm, the flange resting on

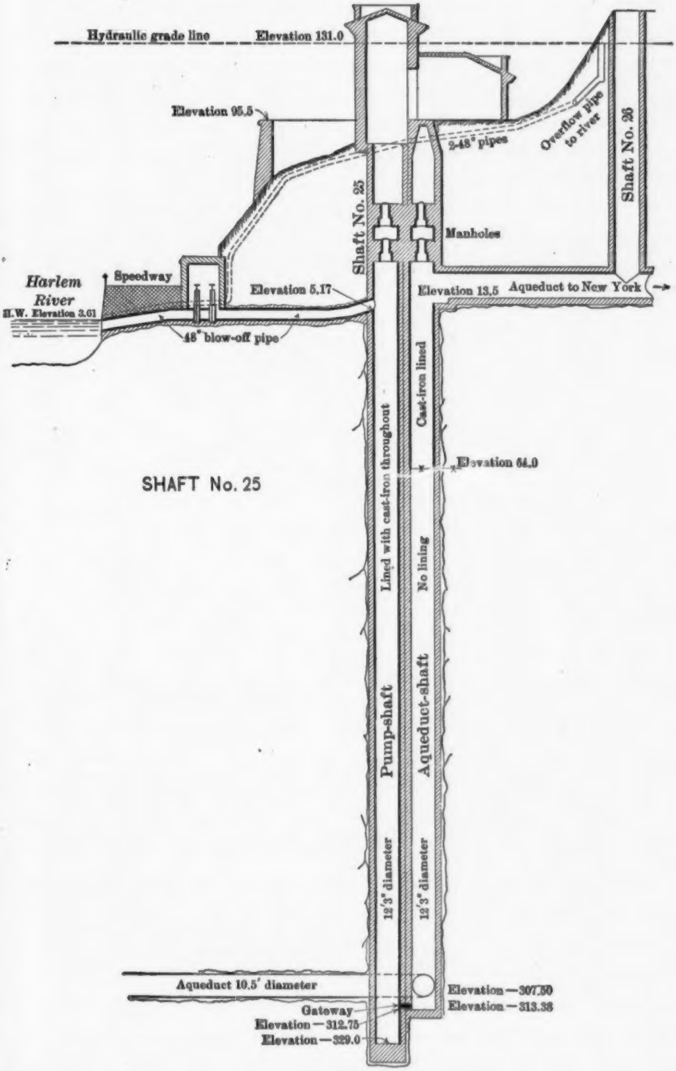

Hydraulic grade line
Elevation 131.0
Elevation 95.5
2-48" pipes
Overflow pipe to river
Shaft No. 26
Shaft No. 25
Manholes
Harlem River
H.W. Elevation 3.01
Speedway
Elevation 5.17
Elevation 13.5
Aqueduct to New York
48" blow-off pipe
Cast-iron lined
Elevation 64.0
SHAFT No. 25
Lined with cast-iron throughout
No lining
Pump-shaft
Aqueduct-shaft
12'3" diameter
12'3" diameter
Aqueduct 10.5' diameter
Elevation —307.50
Elevation —313.38
Gateway
Elevation —312.75
Elevation —329.0

FIG. 2.

its upper side. A similar pipe was placed in each hole of the lower diaphragm, the flange being bolted to the iron lining of the underside thereof. A short length of pipe, of the same inside diameter and with a hub on each end, connected the two pipes of each hole. Perfectly tight joints were made with lead. The utmost care had to be taken in pouring the lead as there was a great deal of moisture in the space between the diaphragms.

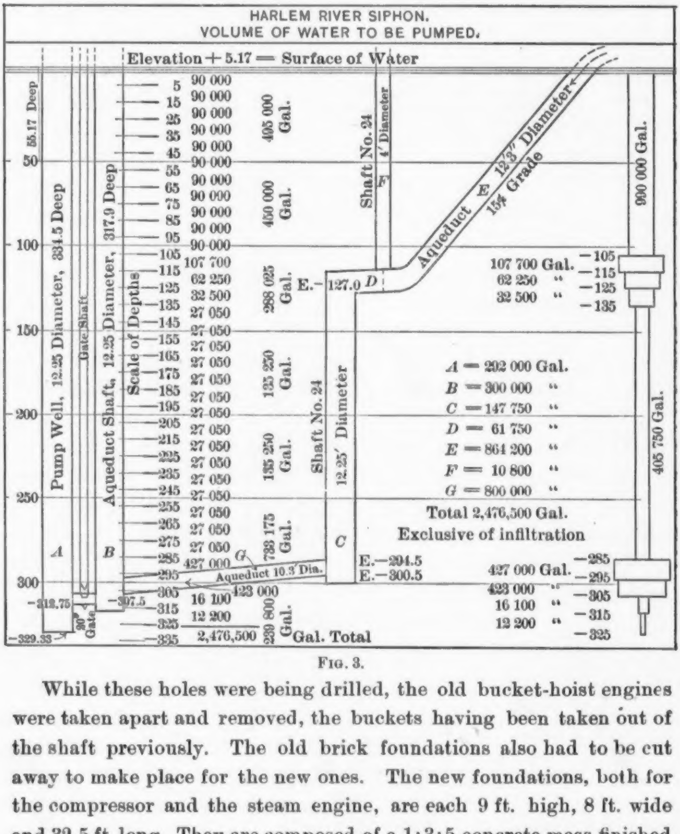
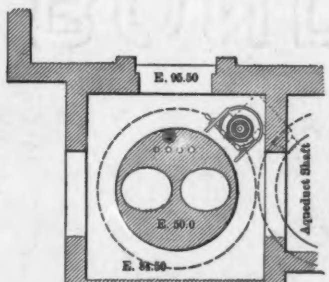


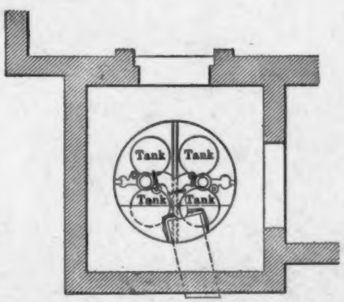
Fig. 3.

While these holes were being drilled, the old bucket-hoist engines were taken apart and removed, the buckets having been taken out of the shaft previously. The old brick foundations also had to be cut away to make place for the new ones. The new foundations, both for the compressor and the steam engine, are each 9 ft. high, 8 ft. wide and 32.5 ft. long. They are composed of a 1:3:5 concrete mass finished with a 1-ft. granite coping stone over the entire top. The foundation of the auxiliary compressor is also of concrete, with granite coping stones.

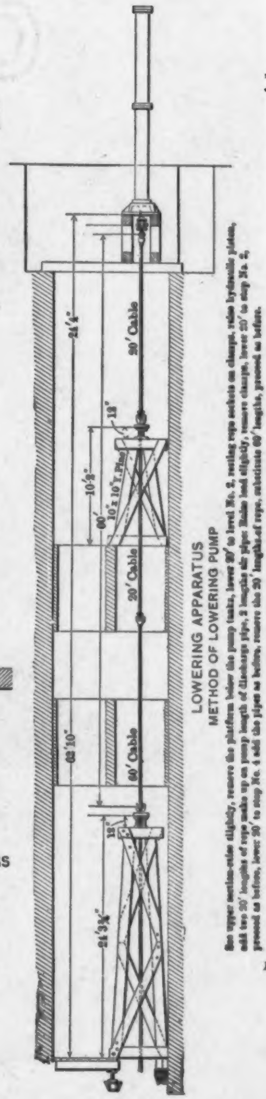
PLATE I.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 988.
DIAMANT ON
PNEUMATIC PUMPING PLANT.

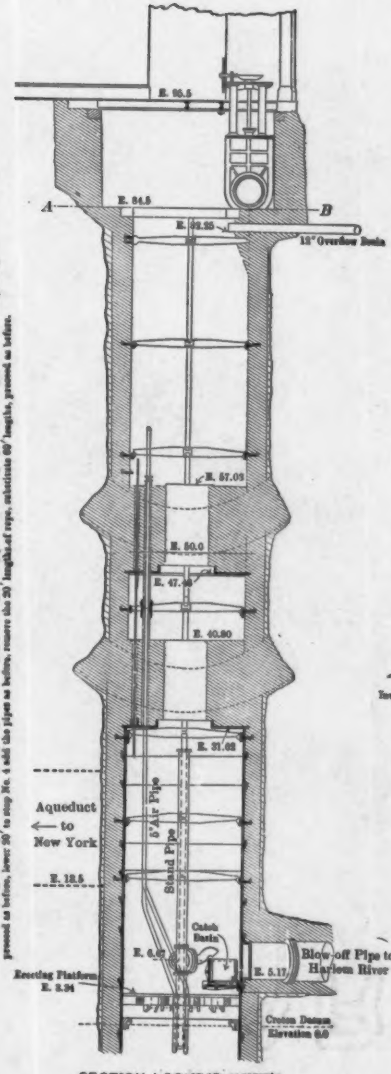


SECTION THROUGH A-B

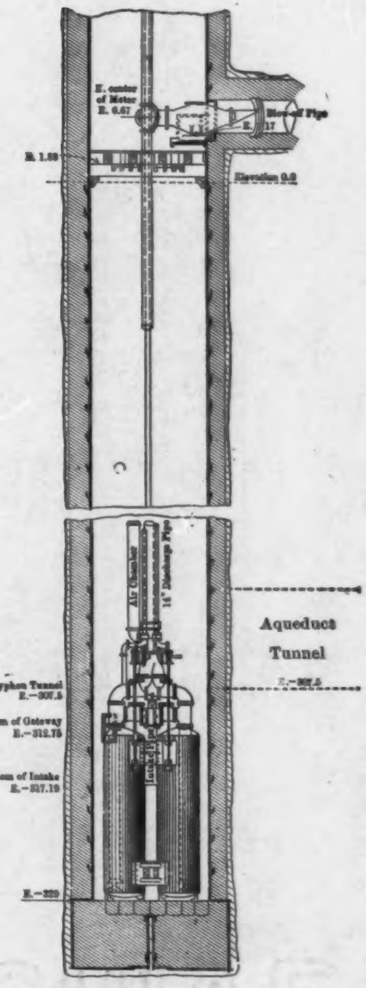


SECTION BELOW MANHOLE



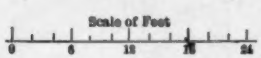


SECTION LOOKING NORTH



SECTION LOOKING NORTH

SECTIONS OF SHAFT No. 25
SHOWING
INSTALLATION OF TANKS AND FITTINGS
FOR
PNEUMATIC PUMPING PLANT



The different parts of the system having arrived, the first pair of tanks was taken into the engine-house and placed in proper position near the top of the shaft at Elevation 84.5. The second pair was then placed also. These tanks are 17.5 ft. high and have an inside diameter of 4 ft. 2 in. A cage, operated by a small Otis steam engine, carried the men from the top of the shaft down to the blow-off at Elevation 5.17. This cage could be shifted to pass through the north or south opening as necessity required. With the four tanks, a clearance for the elevator cage, a 36-in. water main and gate, and the lifting machinery for the connecting gate between the aqueduct- and pump-shafts, there was very little room to spare.

Fig. 1, Plate II, is a front view of the tanks and fittings assembled at Elevation 84.5, before being taken apart to be lowered to the erecting platform. The photograph shows one pair of tanks, the manner in which they are connected, the intake pipe with the 10-in. check-valves admitting water into the tanks, the 14-in. discharge pipes with 10-in. check-valves opening outward, the 5-in. air-pipes, and the I-beams and hangers for lowering the tanks. The discharge pipe passes into each tank within about 6 in. of the bottom, a cone at this point guiding the water from the tank into the discharge pipe (see Plate I). Near the top of the inlet pipe is a cast-iron groove which is to slide along the old bucket-guides in the shaft. Grooves similar to this are on the plates connecting the tanks near the bottom, and also on the plates in the back of the tanks. The I-beams and hangers to which the wire cables of the lowering apparatus are attached can be seen near the extreme top of the photograph.

On top of the Y, connecting the discharge pipes of the two tanks, a T was placed, to which, by means of an elbow, an air chamber was fastened to prevent water ram. A $\frac{3}{8}$ in. pipe, with a check-valve, leads from this elbow to the top of the shaft, being used to charge the air chamber. At this point, also, the discharge pipe and the two air-pipes were fitted with swivel joints, so that, even if the tanks did not rest perfectly level on the bottom, the pipes could be carried up vertically, by means of these joints, which were perfectly tight.

The 14-in. discharge pipes are rolled-steel tubes with cast-steel flanges. These were shrunk on the tubes, and the ends of the latter were upset. The pipes were delivered in this condition, but, as the upset ends projected from $\frac{1}{8}$ to $\frac{1}{2}$ in. beyond the faces of the flanges,

this part had to be removed, otherwise no tight joints could have been made. To return the pipes to the foundry would have caused the loss of too much time, therefore a lathe was rigged up outside of the engine-room. Sand was strewn over the space to be used, some iron-grating floor-plates from the shaft-house were embedded in the sand, and several pieces of the coping stone of the old engine foundation were placed on top of the plates. Two 15-in. I-beams, 20 ft. long, with a space of about 4 in. between them, were laid on top of the stones and fastened firmly by steel rods passing down to the grating plates. The I-beams were leveled carefully, and the 14-in. pipes, being laid on top, were thus also level. The pipes were held fast by a V-shaped clamp at each end. A chuck holding the cutting tool was geared to a shaft which was revolved by being belted to a small vertical steam engine. The tool was fed automatically into the flange to be faced by means of a star wheel which, at each revolution of the chuck, would strike one of its prongs against a projecting board, thus causing the tool to cut deeper. This apparatus proved to be very efficient, as the faces of the flanges were made absolutely at right angles to the axis of the pipe, thus ensuring a perfectly straight column when the lengths were bolted together. There were thirty-two pieces to be faced on each end, and the entire work was completed in 11 days.

While this was being done, men were engaged in placing an erecting platform, just below the blow-off. This consisted of brackets fastened to the east and west sides of the shafts with bolts let into the iron lining. On these brackets was placed a 15-in. I-beam, which was bolted down. Resting upon this beam, and also upon brackets fastened to the north and south sides of the shaft, were placed 12-in. timbers, over which a 3-in. plank flooring was fastened. All drilling for stud-bolts was done with a pneumatic drill, the air being supplied by a small Rand Drill Company's compressor on top of the shaft.

Just before the erecting platform was completed, there occurred the only accident during the entire installation. Fortunately, this was attended with no serious results. In order to lower the tanks, it was necessary to remove the catch-basin into which the buckets of the old system discharged. The bottom plate is shaped like a segment of a circle, with a chord of 11 ft. 6 in., and a rise of 3 ft. 10 in., the

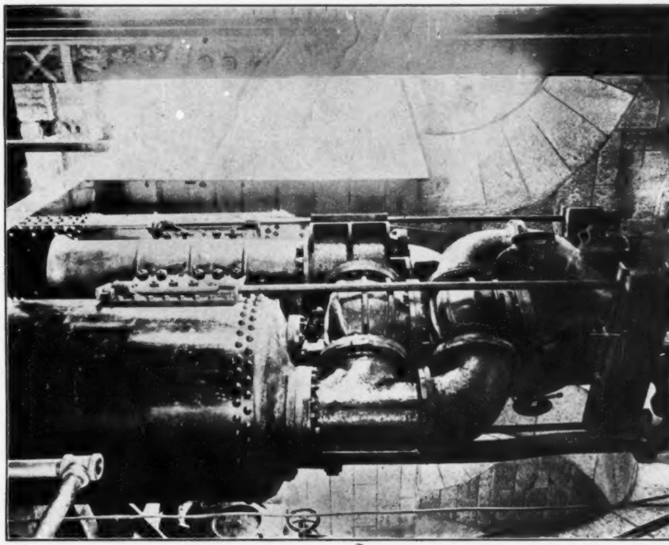


FIG. 1.—FRONT VIEW OF TANKS AND FITTINGS FOR PNEUMATIC PUMPING PLANT.

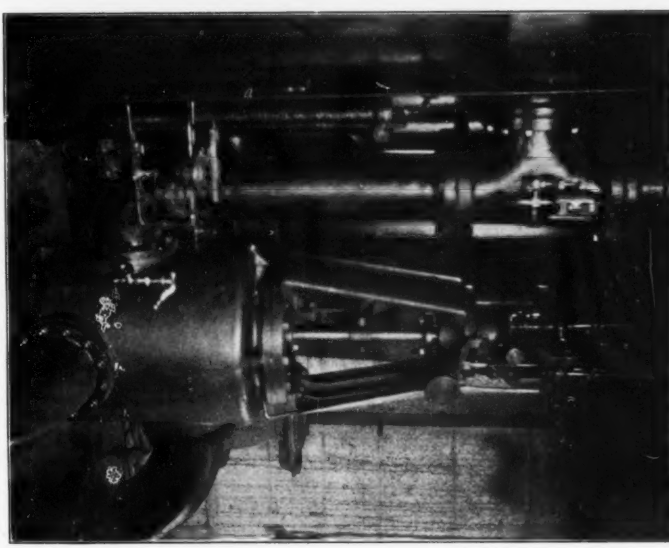


FIG. 2.—AUTOMATIC SWITCH AND CONNECTIONS FOR PNEUMATIC PUMPING PLANT.

radius of the arc being 6 ft. 1¼ in. The weight of the plate was about 2 000 lb. Two 8 by 12-in. holes had been cut into it some years ago. Two workmen were on top of the plate passing a chain through one of the holes, when it slipped from its bearings, tipped over, and dropped to the bottom, a distance of 333 ft. The men were thrown into the water, but, with the exception of some bruises, were not injured seriously.

Before lowering the tanks, this plate had to be recovered. Accordingly, the writer, with an assistant, sounded every foot of the bottom of the shaft with a steel tape and lead weight, and was fortunate enough to locate one of the 8 by 12-in. holes. A chain with a hook at the end was fastened to the elevator cable, lowered to the bottom, and guided from the erecting platform by a ribbon tape. The hole, located by previous soundings, was again found, and, after two or three trials, the plate was hoisted to the surface. A small corner broken off was the only damage the plate sustained.

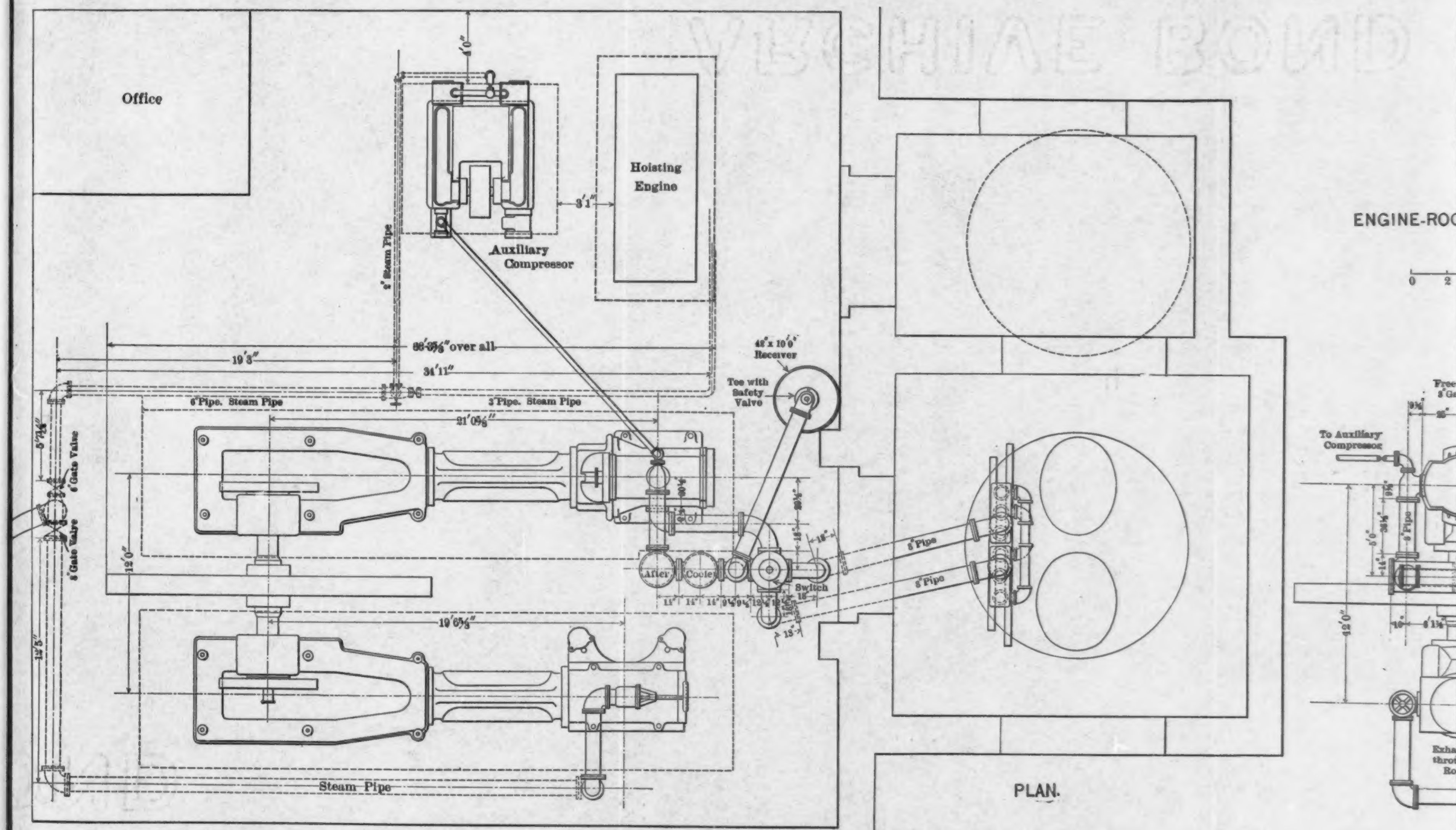
As the soundings indicated some silt at the bottom of the shaft, sixty bags of sand were dumped into the shaft, the bottom being thus fairly leveled.

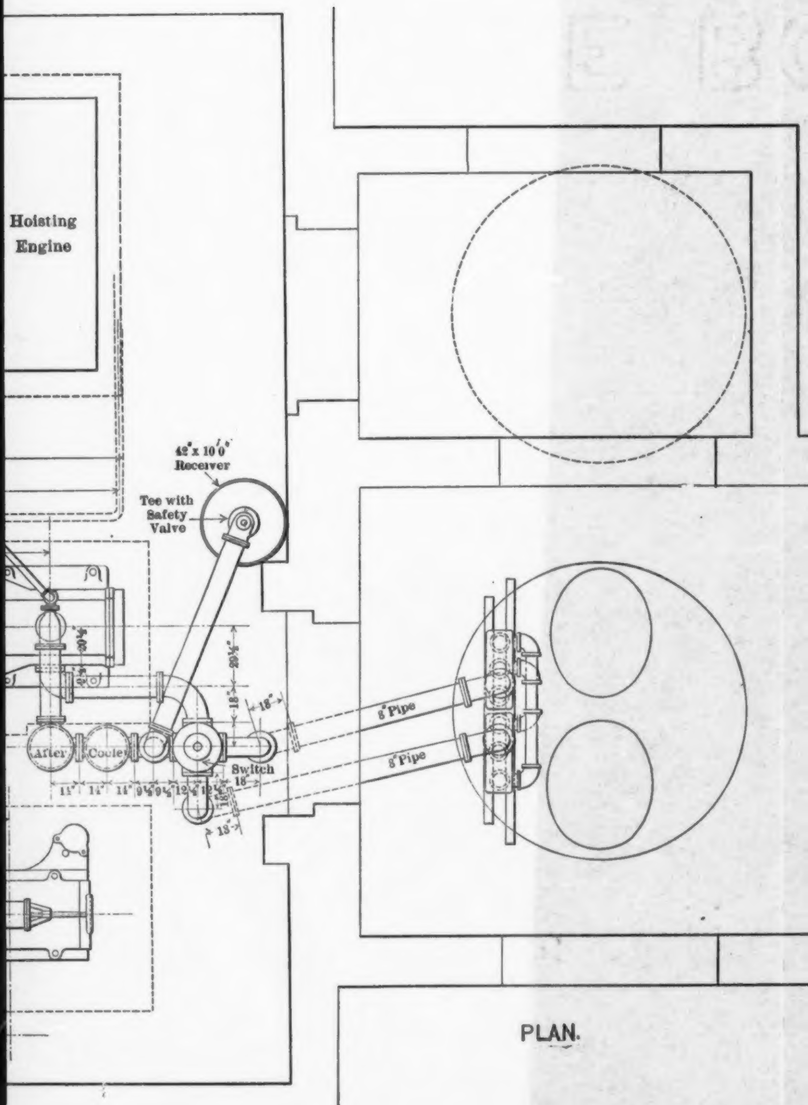
The first pair of tanks and fittings, which had been assembled at Elevation 84.5, was now taken apart, preparatory to being lowered to the erecting platform. The elevator cage was hoisted out of the shaft, so that its cable could be used. Each tank was first lowered to the top of the first diaphragm with a block and fall. The elevator cable was then attached and the tank was lowered to the erecting platform. The tank was hung with such exactness that it passed through the manhole without binding, although there was only about ¼ in. clearance. The T's and Y's, and other parts having been lowered, the first pair of tanks was again assembled, and placed in the exact position in which they would have to be lowered to the bottom.

The first pair of tanks being out of the way, the hydraulic lowering apparatus was set up in the south half of the shaft at Elevation 84.5 (see Plate I). This apparatus consists of a cylinder with a plunger having a stroke of about 21 ft., a balanced elevator-valve and pressure pump, two A-frames on top of the upper diaphragm and two on the erecting platform. In the timber bents, 10 by 10-in. beams were used. Plow-steel wire cables were used, and were fitted with

sockets at each end. Their breaking strain was 98 tons. A 60-ft. length of cable reached from the lifting I-beams of the plunger to the holding I-beams above the tanks, the plunger being about 1 ft. from the top of its stroke. Two cables were used for each pair of tanks. The plunger was now raised as high as it could go, the tanks thus being raised about 1 ft., and the planks and beams on which they had been resting were removed. The water in the hydraulic cylinder having been allowed to exhaust, the tanks were lowered 20 ft. The A-frame on the upper diaphragm had been placed in position, and the sockets of the 60 ft. cables rested on clamps which were now bolted on. These clamps in turn rested on I-beams on top of the bents. While the whole weight rested on these A-frames, the pins, which held the sockets of the cables to the lowering I-beams of the plunger, were removed, the plunger was again raised to near the top of its stroke, and the longer A-frames on the erecting platform were placed in position. A 20-ft. length was added to each cable, a length of 14-in. pipe to the discharge pipe, a length of 5-in. pipe to each of the air-pipes, and a length of ⅜-in. pipe to each of the charging pipes of the air chambers. Each joint was tested under an air pressure of 150 lb., as were also the tanks and valves before lowering, to insure perfect tightness of all joints. The load was now raised slightly, the clamps removed, and the tanks lowered 20 ft. The procedure being the same, the tanks were lowered another 20 ft., this time, however, the clamps rested on the I-beams of the lower A-frames. The three 20-ft. lengths of cable were now taken out and replaced by a 60-ft. length, and the cycle again started. In this manner the first pair of tanks was safely lowered to the bottom of the shaft, a depth of 332 ft. Pipe-guides or stays were fastened to the pipes every 60 ft., the ends of the stays sliding along the old bucket-guides. The total weight lowered was estimated at about 40 tons. The elevator cage was now shifted to the other side of the shaft, as was also the hydraulic lift, and the second pair of tanks was lowered in the same manner as the first. All parts before going down were painted both outside and inside with two coats of "Nobrac" paint.

The top of the discharge pipe of the second pair of tanks was about 4 in. above that of the first pair. Short lengths of discharge pipe added to each brought them to the same level. To this discharge pipe was added a T-piece. A Y connected the T-pieces, and a 20-in. goose-

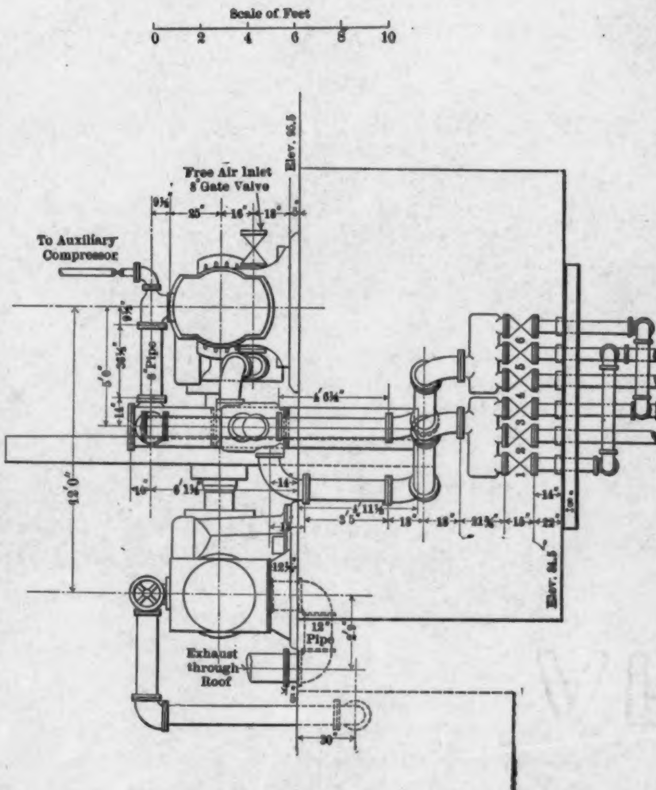

Office
Hoisting Engine
Auxiliary Compressor
2" Steam Pipe
3'1"
86'3⅝" over all
19'3"
34'11"
6" Pipe. Steam Pipe
3" Pipe. Steam Pipe
21'0⅝"
6" Gate Valve
12'0"
19'6⅝"
Steam Pipe
48"x10'0" Receiver
Tee with Safety Valve
After
Cooler
Switch
5" Pipe
5" Pipe
To Auxiliary Compressor
5" Pipe
12'0"
PLAN.


Hoisting Engine
42" x 10'0" Receiver
Tee with Safety Valve
After Cooler
Switch
8" Pipe
8" Pipe

PLAN.

PLATE III.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 988.
DIAMANT ON
PNEUMATIC PUMPING PLANT.

ENGINE-ROOM AND PUMP SHAFT.



ELEVATION.

neck, of galvanized-iron pipe was bolted to the Y. This pipe discharged into a catch-basin at the entrance of the blow-off, the bottom plate being the same one that fell to the bottom of the shaft, the sides being smaller than those of the old one. Cover-plates were bolted to the top of the two T-pieces. The four 5-in. air-pipes were now carried up to Elevation 84.5. Glands, through which these pipes passed, were bolted to the flanges of the iron lining of the holes through the upper diaphragm, so that, if the covers of the manholes were put on, the water could not pass between the air-pipes and the lining of the holes. When the shaft was built, a 4-in. pipe from the bottom of the lower diaphragm to a point 1 or 2 ft. above the hydraulic grade, served as an air-vent when the manhole covers were on. This pipe had been removed, above the upper diaphragm, and the two $\frac{3}{8}$-in. pipes were carried through this 4-in. opening to the top. This 4-in pipe was afterward replaced.

At Elevation 84.5 the four 5-in. air-pipes were connected with two manifolds, and from each of the manifolds an 8-in. air-pipe led to the switch. By means of these manifolds, any two tanks could be cut out of service and the pumping done with the other two. (See elevation of general plan, on Plate III.) The two $\frac{3}{8}$ in. pipes from the air chambers were connected by a Y, and led to the 8-in. air-pipe from the after-coolers to the switch. A $\frac{3}{8}$-in. pipe from the same point, with a pin-valve to allow air to leak into it, was hung down into the shaft, within 6 ft. of the bottom, passing through the 4-in. opening through the diaphragms. This shows the pumping level and the pressure due to the head of water in the shaft. An 8-in. pipe carries the return air from the switch to the compressor on the side opposite the free-air valve.

The switch consists of a plunger, with a stroke of $6\frac{5}{8}$ in., operated by a piston moving in a smaller cylinder. The air is introduced into this smaller cylinder by a valve which depends for its action on a piston in a small cylinder, which, in turn, is caused to move by the action of a disc-valve. (See Fig. 2, Plate II.) The disc or diaphragm is 6 in. in diameter, with a movement of $\frac{3}{32}$ in., and consists of two thin sheets of bronze and one sheet of steel. A $\frac{3}{8}$-in. pipe conveys the return air from a point near the top of the cylinder of the plunger to one side of the disc-valve, and the $\frac{3}{8}$-in. pipe, which shows the pumping level and pressure due to the head of water, leads to the other side

of this valve. This latter $\frac{3}{8}$-in. pipe, connected with the 8-in. pipe from the after-coolers to the switch, receives air through a pin-valve, and is also piped to a gauge on the gauge-board, so that the pumping level and the pressure due to the head of the water can be seen at a glance. A small reservoir on this line gives a constant supply of air. The small, return air-pipe is also piped to a gauge, showing the return pressures. The difference between the pressure due to the head of the water in the shaft, which for the same levels is constant, and the return pressure (which is varying constantly, and drops to zero when the switch acts), causes the disc-valve to move. As the operation of the switch requires an air pressure of only about 50 to 60 lb. per sq. in., a $\frac{3}{8}$-in. pipe from the 8-in. compressed-air pipe conveys the air through a reducing valve to the cylinder on top of the plunger, and to the piston of the small cylinder operated by the disc-valve. A reservoir on this line also ensures constant pressure. This $\frac{3}{8}$-in. pipe also leads to a dial on the gauge-board, showing the pressures required to operate the switch. The disc-valve moving, due to the difference between the pressure caused by the head of the water in the shaft and the return pressure, allows air to enter the small cylinder above it, the piston moves, the valve controlled by this piston motion allows air to enter above or below the piston in the cylinder above the plunger, the plunger acts and the air is sent alternately from one 8-in. air-pipe into the other, one of these 8-in. pipes always serving to return the air through the switch to the compressor. (See Plate III.) Provision is also made for operating the actuating valve by hand.

The auxiliary compressor was set up, the large compressor and engine were adjusted, the piping was completed between the auxiliary compressor, the large compressor, the switch after-coolers, and the receiver, and the plant was ready for operation. Before pumping, all joints were tested as to tightness.

The action of the plant is as follows: The large compressor is first started; the exhaust valve being closed, it requires about 312 revolutions of the fly-wheel to charge the system, the free air being compressed to 150 lb. per sq. in. The free-air valve is now closed and the switch thrown over by hand. The compressed air passes from the compressor through the two after-coolers, into a receiver supplied with a safety valve, and also through the switch through one of the 8-in. pipes, through its manifold into the 5-in. air-pipes and into one pair of tanks.

The air entering this pair of tanks forces the water through the discharge pipes and empties the tanks. As soon as this occurs, the return pressure from the other pair of tanks being less than the pressure due to the head of the water in the shaft, the actuating valve of the switch acts, the plunger moves, compressed air enters these tanks, while the air from the other pair is returning through the switch into the compressor to be used over again, and so on. A cycle consists of the number of revolutions of the fly-wheel necessary for the compressor to empty one pair of tanks to the point of starting to empty the other pair. The number of revolutions per cycle varies for different pumping levels, but is constant for the same level. If there are too many revolutions in charging the machine, or if there are too many revolutions per cycle, the air follows the water through the discharge pipes, and thus the system loses the air, necessitating the opening of the free-air valve of the compressor and recharging. After the plant has been working, a certain amount of air is lost, and in order to keep up the proper number of revolutions per cycle the auxiliary compressor is started, and furnishes the air to keep the system working efficiently.

Before the final test, many trials were run. Indicator diagrams of the Corliss engine were taken, and adjustments were made. The goose-neck discharge pipe was removed, and a 20-in. Gem meter placed in its stead. The cover-plates were taken off the T's, and a 20-ft. length of 14-in. galvanized-iron pipe was added to each, so that no air would pass through the meter, but would escape through these pipes. (See Plate III.) Many over charges took place before the proper number of revolutions per cycle for different pumping levels was determined.

The 20-in. Gem meter consists of a system of helicoids formed around a vertical central hub, revolving in a cylinder slightly greater in length, and having a diameter just large enough to receive it. A screen at the lower end of this cylinder serves to keep large objects from entering the meter. The axle of the hub is geared to the meter register, which contains six figures and reads thousands of gallons.

As it was necessary to test the accuracy of this meter, before using it, to determine the efficiency of the pneumatic pumping plant, F. W. Watkins, M. Am. Soc. C. E., Division Engineer of the Aqueduct Commissioner's Engineering Department, assisted by the writer, made a

test of the meter at the testing plant of the National Meter Company, in South Brooklyn.

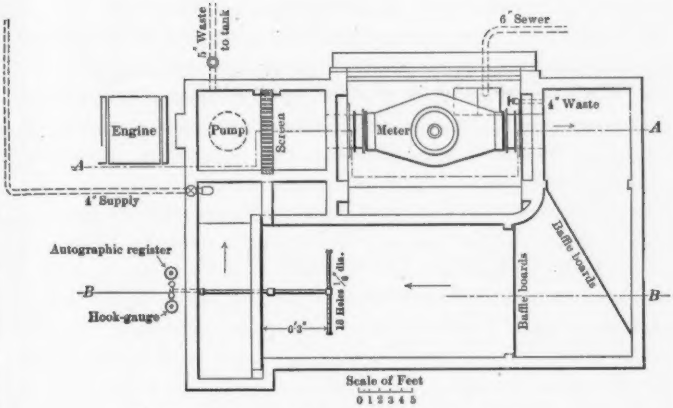
The test consisted of a comparison of the meter register records with weir measurements of the same volume of water. The water to be measured was elevated by a centrifugal pump operated by a Nash gas engine to a height which gave a head sufficient to force the desired amount through the meter. The water passed from the pump, through a screen, into a small forebay, thence through the meter into the L-shaped weir chamber. The base of the L is about 8 ft. long and 8 ft. wide, and the long side, constituting the main weir chamber, is 33 ft. long, 12 ft. wide and 6 ft. deep below the level of the weir crest. Baffle boards, placed in the angle of the L, serve to break up any eddies which may form. The water flowing over the weir drops into the pump-well, and the cycle is again started. (See Fig. 4. Figs. 4 and 5 were furnished by John H. Norris, M. Am. Soc. M. E., Assistant Engineer, National Meter Company, whom the writer takes this opportunity of thanking for his courtesies during the test.)

The weir notch is of cast-iron plates, the plates forming the sides of the notch being adjustable, so that any length of weir, up to 8 ft., can be obtained. The crest was formed by beveling the down-stream face at an angle of 45°, leaving a truly planed edge $\frac{1}{4}$ in. thick, the vertical sides having a similar bevel. The distance from the bottom of the weir chamber to the crest is 6 ft.

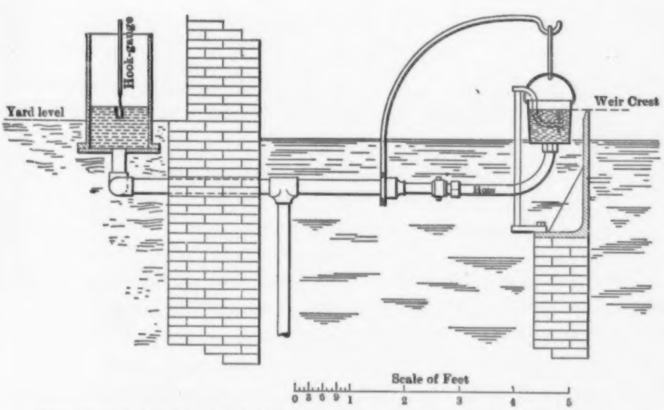
The apparatus for measuring the head on the weir consists of two 12-in. cast-iron pipes set on end just outside of the catch-basin, one containing a float for the autographic record, the other the movable hook-gauge.

These pipes are connected by a 2-in. pipe from which a 2-in. pipe leads through the wall of the catch-basin, with a valve at the other end. Another 2-in. pipe runs from this pipe to the bottom of the catch-basin, makes a right-angle bend, thence, parallel to and about 6 in. above the floor, it runs into the weir chamber and connects with a 2 in. pipe at right angles to it, parallel to the weir crest, and about 6.25 ft. from the weir plate. This latter pipe was perforated with eighteen holes, each $\frac{3}{16}$ in. in diameter. (See Fig. 4.)

Before starting the test, the relation of the hook-gauge and the autographic float-gauge to the weir crest was determined as follows: A fixed hook-gauge was fastened a few inches in front of the weir,



TESTING PLANT FOR LARGE METERS
FIG. 4.



CHECKING HOOK OF LARGE WATER METER TESTING PLANT
FIG. 5.

and, by a spirit level, its point was adjusted exactly to the elevation of the weir crest. (See Fig. 5.) A bucket, with a rubber hose attached to the bottom, was hung over this fixed hook-gauge, the other end of the hose being attached to the 2-in. pipe leading to the movable hook-gauge, and to the autographic record. Water was poured into the bucket until the surface just covered the point of the fixed hook, when the water rose to the same elevation in the two 12-in. cast-iron pipes. The zero of the movable hook-gauge and the fixed pencil of the autographic gauge were now adjusted to correspond. The autographic gauge consists of a zinc float carrying a brass rod, to which a pencil is attached. Its point presses against a paper wrapped around a wooden cylinder revolving once an hour by clockwork. Another pencil, attached to the frame holding the drum, marks a line corresponding to the elevation of the weir crest, so that the actual heads of water flowing over the weir can be seen at a glance.

These preliminaries being over, the bucket was removed and the test begun. The weir opening was measured by a standard steel rule and was 4.2475 ft. Sufficient water from the city main was allowed to run into the catch-basin, the pump was started, and the water began to circulate. In order that the wind might not affect the test, the weir chamber was covered with boards.

The Francis formula, with Hamilton Smith's correction, in the form of

$$Q = 3.29\left(L - \frac{H}{10}\right) H^{\frac{3}{2}},$$

was used to calculate the quantity of water passing over the weir. In this formula

Q represents cubic feet of water per second;
L represents the length of the weir, in feet;
H represents the head, in feet.

The velocity of approach was so small that it did not enter the calculation at all. The heads scaled from the autographic record checked very closely with the hook-gauge record. Table 1 is a summary of the tests, and is taken from the report of Major Watkins to William R. Hill, M. Am. Soc. C. E., then Chief Engineer of the Aqueduct Commission.

TABLE 1.—SUMMARY OF METER TESTS.

Test.	Head, in Feet.	Gallons per minute.		Percentage, meter to weir.
		Meter.	Weir.	
First	0.296	1 002	1 003	99.90
Second	0.491	2 140	2 133	100.33
Third	0.603	2 905	2 895	100.34

These tests proved the meter to be very accurate and consistent for different heads, and it was recommended by Major Watkins as the standard measure for the pneumatic pumping plant at Shaft No. 25.

As it was impossible at that time to shut down the aqueduct, so that the siphon could be actually emptied, it was decided to pump at different water levels in the pump-shaft. The water was first pumped out through the blow-off pipe, until its surface was about 50 ft. below it, when the gate was closed far enough to allow only the leakage into the shaft to pass through, the remaining water running back into the shaft. After pumping at this level for an hour, the gate was opened and the water pumped down to 125 ft. below the blow-off, when the gate was closed down again to allow only the leakage to run off. In like manner the plant was tested for levels 175, 225 and 300 ft. below the blow-off.

The average volumes pumped per minute, as indicated by the Gem meter, were as follows:

At 88 ft. below the blow-off..........6 290 gal. per min.
" 125 " " " " "..........6 020 " " "
" 175 " " " " "..........5 220 " " "
" 230 " " " " "..........4 286 " " "
" 298 " " " " "..........2 180 " " "

A table showing the details of these tests has been filed for reference in the Library of the Society.

It had also been agreed to run an endurance test of 12 hours, pumping at a level about 175 ft. below the blow-off, but, owing to the dismantling of several boilers, sufficient steam could not be obtained and the test was postponed for several weeks. In the meantime, the machinery was overhauled; a revolution counter was placed on the

auxiliary compressor, and a small pump lubricator attached to the switch-plunger cylinder. A small steam pump was also connected with the line of water pipe leading from the 36-in. pipe to the water jackets on the large and auxiliary compressors and also to the after-coolers, as previous to this there was not sufficient water to keep the air properly cooled.

In conclusion, the writer wishes to express his thanks to J. Waldo Smith, M. Am. Soc. C. E., Chief Engineer, and to Major F. W. Watkins, Division Engineer, for their kind interest in the preparation of this paper.

DISCUSSION.

ELMO G. HARRIS, M. AM. SOC. C. E. (by letter).—Mr. Diamant has well presented the extraordinary conditions under which this particular pump must act, and the many difficulties accompanying its erection. It may be of interest to describe briefly the special features involved in the operation of this system of pumping and give some of the mathematics involved in proportioning a plant properly. Mr. Harris.

The general principles involved are easily stated and readily understood. The special features are:

First.—The system, once charged with air, is closed to the atmosphere; the one charge being forced alternately into one tank while being drawn out of the other. Hence the energy of expansion (or compression) in the air is not lost, as in the common forms of direct air-pressure pumps. The only recognizable losses in the system, outside the air compressor, are: Expansion in the low-pressure air-pipe immediately after switching, friction in the air-pipes, conduction of heat out of the air-pipes (or absorption, which is a gain), and leakage of air. These are not capable of formulation—except the first and second, and these only approximately.

Second.—The automatic switching device, by which the air is alternately exhausted from one tank and delivered to the other. This is accomplished by utilizing the difference of pressure inside and outside the tank from which air is being exhausted. For convenience, the mechanism utilizing this is placed in the compressor-room. If the tanks are near the free surface of the water supply, there will, at the time the switch should act, be suction in the air-pipe leading to the tank from which air is being exhausted, and which is filling with water. This suction, of course, extends throughout the air-pipe, and is utilized in the compressor-room, in conjunction with free air pressure, to operate the switch.

In case the tanks are deeply submerged, as in the pump under discussion, suction will not occur, but the tanks will fill under pressure. In this case the pressure outside the tank is communicated to the compressor-room by a "dip" pipe, which descends to the tank level, and through which air, in minute quantity, is forced continuously, thus registering within the compressor-room the head outside the tanks, while the main air-pipe registers the pressure within the tanks; as before said, when the pressure within the tank is less than outside, the switch acts. The switch is made adjustable; that is, it can be made to act at any desired difference of pressure. Evidently, the details of the switch can be varied without limit. When suction occurs before the switch acts (which is usually the case) the leakage is re-

Mr. Harris. placed automatically by an adjustable valve placed outside a check-valve opening into the intake pipe between the switch and the compressor.

Another method of operating the switch is by a mechanism which acts at a prescribed number of strokes of the compressor —the number being that necessary to complete a cycle.

It may be remarked, before going further, that the submergence of the tanks is not necessary.

With the development of this system of pumping, many problems have been presented for solution, some purely mechanical, while others require a mathematical analysis. The latter have proved very interesting and instructive.

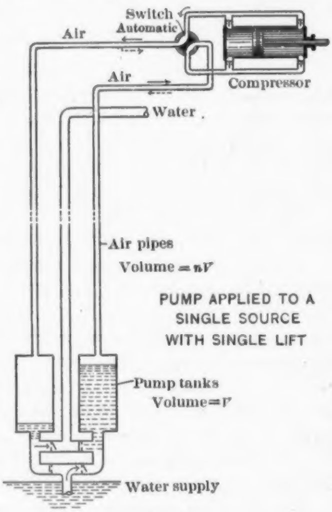


FIG. 6.

In the process of such analysis, it will be necessary to use the following symbols. Though the analysis may be considered intricate, the final formulas are unexpectedly simple and easy of application:

Let P_o = Delivery pressure—a constant—in pounds per square inch;

P_1 = Pressure throughout the system immediately after switching;

P_x = Pressure of air entering compressor— a variable;

V = Volume of one pump tank—a constant—in cubic feet;

V_y = Volume of air in delivering tank at pressure, P_o — a variable;

$n\ V$ = Volume of one air-pipe;

p_1 = Pressure at which water begins to enter tank from which air is being exhausted;

p_o = Lowest pressure reached (this occurs just before switching);

q_a = Effective volume, intake of compressor, in cubic feet per second;

q_w = Average water delivery, in cubic feet per second;

Q = Total volume taken into compressor, while working pressure down from P_1 to p_1, or approximately P_o to p_1 in any case and approximately P_o to p_o when tanks are near surface of water supply;

Mr. Harris.

$R_o = \text{ratio } \frac{P_o}{p_o};$

$R_1 = \text{ratio } \frac{P_1}{p_1}.$

All pressures are "absolute," that is, gauge pressure + 14.7 lb.

Compressor Capacity ($= q_o$).—The first problem is to find the necessary intake capacity of the compressor. In this, fortunately, the problems of work and temperature inside the compressor need not be considered, and, therefore, in the analysis, the temperature of the air may be considered as constant, though it will be necessary, finally, to apply a coefficient to provide for the effect of expansion due to the heating of the air as it passes through the hot intake valves.

Assume that a small volume, $d\,Q$, of air at the pressure, P_x, is taken out of the exhausting tank and forced into the delivery tank, where the pressure is P_o, and its volume is $d\,V_y$, then, by the law that the pressure multiplied by the volume is constant:

$$P_x\,d\,Q = P_o\,d\,V_y;\ \text{or } d\,Q = \frac{P_o}{P_x}\,d\,V_y \ldots\ldots\ldots\ldots\ldots \text{I}$$

Also, by the same law, the sum of the product of the pressure multiplied by the volume must be constant, since the quantity (or mass) of air in the system does not change. When one tank is full of water, and its air-pipe is full of air at the pressure, p_o, the other tank and air-pipe must be full of air at the pressure, P_o. Under this condition, the sum of the products is

$$P_o\,V(1+n) + p_o\,V\,n.$$

At any other time the sum of the products is

$$P_x\,V(1+n) + P_o\,(V_y + n\,V).$$

Hence, $P_o\,V(1+n) + p_o\,n\,V = P_x\,V(1+n) + P_o\,(V_y + n\,V)$.II

To simplify, put $p_o = \frac{P_o}{R_o}$ and Equation II reduces to

$$\frac{P_o}{P_x} = \frac{V(1+n)}{V\left(1+\frac{n}{R_o}\right) - V_y} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{III}$$

Substitute Equation III in Equation I, and

$$d\,Q = V(1+n)\,\frac{d\,V_y}{V\left(1+\frac{n}{R_o}\right) - V_y}.$$

Integrating between the limits, $V_y = V_1$ and $V_y =$ zero, there results:

$$Q = V(1+n)\log_e \frac{V\left(1+\frac{n}{R_o}\right)}{V\left(1+\frac{n}{R_o}\right) - V_1} \ldots\ldots\ldots\ldots \text{IV}$$

Let V_1 represent the volume of air in the delivery, or high-pressure tank, when water begins to enter the other; that is, when the pressure in the other tank has dropped to p_1; this marks a change in the opera-

Mr. Harris. tion; see Fig. 7. Just at this period there must be enough air, at the pressure, p_1, in the volume, $V(1+n)$, to fill the space, $V - V_1$, at the pressure, P_o, in the other tank, and its own air-pipe at the pressure p_o. Hence the equation:

$$p_1 V(1+n) = P_o(V - V_1) + p_o n V \text{..............} \quad \text{V}$$

or,

$$P_o V_1 = V[P_o - p_1 + n(p_o - p_1)].$$

Now, n is a fraction, and p_o and p_1 are small and nearly equal, in practice; hence $n(p_o - p_1)$ can be neglected. Then:

$$V_1 = \frac{V}{P_o}(P_o - p_1) \text{......................} \quad \text{VI}$$

Putting Equation VI in Equation IV, there results:

$$Q = V(1+n)\log._e\left[\frac{1 + \frac{n}{R_o}}{1 + \frac{n}{R_o} - \frac{P_o - p_1}{P_o}}\right]$$

$$= V(1+n)\log._e\left(\frac{1}{1 - \frac{P_o - p_1}{P_o + n p_o}}\right)$$

putting $\frac{P_o}{p_o}$ in place of R_o.

Now, as before stated, $n p_o$ will be quite small, as compared with P_o, and it can be neglected, if desired, to simplify the formulas. Equation VI would then become:

$$Q = V(1+n)\log._e\frac{P_o}{p_1} \text{.................} \quad \text{VII}$$

This gives a simple formula for Q, the volume taken into the compressor while reducing the pressure from P_o to p_1 (in a tank full of air). To be precise, it should now be noticed that the operation begins properly with a pressure, P_1, somewhat less than P_o. This is due to the expansion into the low-pressure pipes just after switching. This pressure, P_1, can be found readily by the condition of the constancy of the sums of the products of the volumes by the pressures. Thus, equating the sums just before and after switching, there results:

$$P_1(V + 2n) = P_o V(1+n) + p_o n V$$

or,

$$P_1 = \frac{P_o(1+n) + n p^o}{1 + 2n} \text{........................} \quad \text{VIII}$$

P_1, thus found, would be put in place of P_o in Equation VII.

The effect of friction in the air-pipe between the tank and the compressor must now be considered.

When the pressure of the intake of the compressor is P_2, that in the tank from which the air is drawn will be greater by the amount lost in friction while passing through the pipe. The equation for this loss is, in form,

Mr Harris.

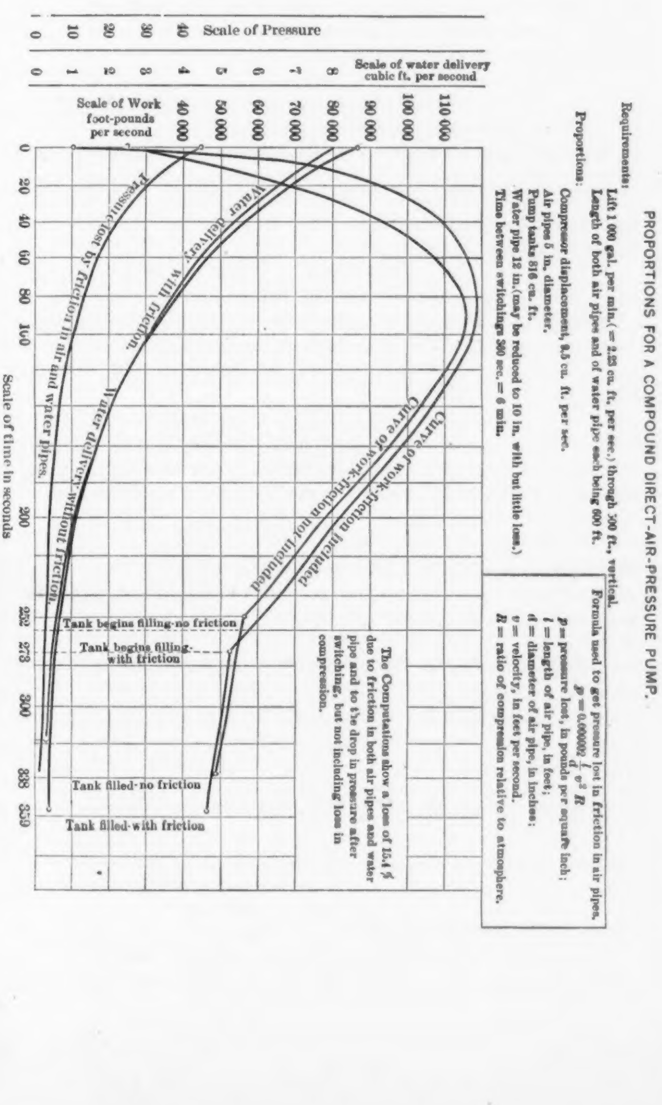


FIG. 7.

Mr. Harris.

$$f = c \frac{l}{d} v^2 R$$

where c is an experimental coefficient. From the best experimental data obtainable, it is found to be about 0.000002, when

f = lost pressure, in pounds per square inch;
l = length of pipe, in feet;
d = diameter of pipe, in inches;
v = velocity of air in pipe, in feet per second;
R = ratio of compression, in atmospheres.

In many rules for computing the loss by friction, the factor, R, is erroneously omitted. In this case, $R = \frac{P_x}{14.7}$, and, therefore, is variable, but in any installation all are constant in the formula except P_x. Then, for simplicity, let

$$\frac{0.000002}{14.7} \frac{l}{d} v^2 = k \text{ .... IX}$$

Then the lost pressure would be $k\ P_x$, and, in Equation II, $P_x(1 + k)$ should be put in place of P_x, but this will in no way change the process by which Equation VII is derived. With this change, Equation VII becomes

$$Q = V(1 + n) \times (1 + k) \log. \frac{P_1}{p_1}.$$

If the compressor takes in a volume, q_a, per second, the time consumed in working the pressure down from P_1 to p_1 is

$$t_1 = \frac{Q}{q_a} = \frac{V}{q_a} (1 + n)(1 + k) \log. \frac{P_1}{p_1}.$$

During the remainder of the time in one cycle, the water is flowing into the tank, following up the air, and keeping it at nearly constant pressure (when the height of the tank is only a few feet); in other words, for every cubic foot of air taken out, a cubic foot of water flows in. Hence, evidently, the time consumed in this last period of the cycle is

$$t_2 = \frac{V}{q_a}$$

and the total time, $T = t_1 + t_2 = \frac{V}{q_a} + \frac{V}{q_a}(1 + n)(1 + k) \log. \frac{P_1}{p_1}$.

If q_w is the average rate of delivery of the water, evidently,

$$q_w = \frac{V}{T}$$

Whence,
$$q_a = q_w \left[1 + (1 + n)(1 + k) \log. \frac{P_1}{p_1}\right] \text{ .... X}$$

which is the desired equation.

In practice, k should not exceed 0.1, and will usually be less. If great precision is to be attempted, Equation X must be solved by a

tentative process, for k is a function of q_a. k may be first assumed as 0.1, to get an approximate value of q_a, whence v in Equation IX, and a closer value of k. This will be sufficiently close for practice. Mr. Harris.

It is probably useless to attempt extreme precision in these computations, on account of temperature changes which cannot be formulated. Hence, as a safe and simple working formula, the following may be used:

$$q_a = q_w \left[1 + 1.1\,(1 + n) \log. \frac{P_o}{p_o}\right] \ldots\ldots\ldots\ldots\ldots \text{X}a$$

p_o will commonly be near atmospheric pressure (or 15), that is, when the tanks are near the surface of the water, but it may be greater or less, according to whether the tanks are submerged or placed above the water. Inspection of Equation X reveals the fact that the greater p_o is, the less will be q_a. For this reason, there is an advantage in having the tanks submerged.

Evidently, if the air is heated by contact with hot surfaces while entering the compressor, the effective intake capacity is reduced. To allow for this circumstance, q_a, as above computed, should be multiplied by $\frac{\tau_2}{\tau_1}$, where τ_1 and τ_2 are the absolute temperatures before and after entering the compressor, respectively.

Maximum Rate of Work.—The compressor capacity having been determined, the next problem in the design of a plant is to find the maximum rate of work for which provision must be made in the steam end of the compressor. The nature of this problem can best be presented by first studying the case of isothermal compression. In this the well-known formula for work, using the symbols heretofore applied, is

$$\text{Work per second} = P_x\, q_a \times \log. \frac{P_o}{P_x} \ldots\ldots\ldots\ldots \text{XI}$$

In this, P_x is variable, and, evidently, the work will be zero when P_x = zero, and again, when $P_x = P_o$ (since log. $1 = 0$), and, by the method of calculus, it is found to be a maximum when log. $\frac{P_o}{P_x} = 1$; that is, when $\frac{P_o}{P_x} = 2.72$.

Note that hyperbolic logarithms must be used in all the foregoing equations as they appear. If common logarithms are to be used, multiply by 2.3.

Inserting the condition for a maximum in Equation XI and reducing to foot-pounds per second, there results:

$$\text{Maximum work} = 52.9\, P_o\, q_a.$$

A curve showing the work by Equation XI is given in Fig. 7. In practice, the curve does not reach zero at either end.

To find the maximum work when temperature changes are consid-

Mr. Harris. ered, one must start with the established formula for work when compression is adiabatic, *viz.*:

$$\text{Work} = \frac{n}{n-1} P_x q_a \left[\left(\frac{P_o}{P_x}\right)^{\frac{n-1}{n}} - 1 \right] \ldots\ldots\ldots \text{XII}$$

where n is the "temperature exponent" and equals 1.41 when no cooling occurs.

By the methods of the calculus Equation XII will be found to be the maximum when $\left(\frac{P_o}{P_x}\right)^{\frac{n-1}{n}} = n$; or when $P_x = \frac{P_o}{n^{\frac{n}{n-1}}}$.

This, inserted in Equation XII, gives

$$\text{Maximum work} = \frac{P_o q_a}{n^{\frac{1}{n-1}}} \ldots\ldots\ldots\ldots\ldots\ldots \text{XIII}$$

When $n = 1.41$ Maximum work $= 62.3\ P_o q_a$ foot-pounds per second.
" $n = 1.25$ " " $= 59.0\ P_o q_a$ " " " "
" $n = 1.00$ " " $= 52.9\ P_o q_a$ " " " "

the last number having been derived by analysis of Equation XI.

As a simple approximate rule, the maximum horse-power rate may be taken as $0.1\ P_o q_a$.

This maximum rate should not be confused with the average.

Efficiency.—The only loss of energy chargeable to this system is that caused by the drop in pressure due to expansion into the low-pressure pipe just after switching. This drop is shown in Equation VIII. The ratio of this change of pressure is $\frac{P_o}{P_1} = \frac{1 + 2n}{1 + n + \frac{p_o}{P_o} n} = r$, for simplicity. The necessary work to restore this pressure is

$$P_o V (1 + n) \log. r,$$

while the useful work done during a cycle is $(P_o - 14.7) V$; that is, the water displaced multiplied by the gauge pressure. Hence

$$\text{Efficiency} = E = \frac{(P_o - 14.7) V}{(P_o - 14.7) V + P_o V (1 + n) \log. r}$$

$$= \frac{1}{1 + \frac{P_o}{P_o - 14.7}(1 + n) \log. r} \ldots\ldots\ldots\ldots\ldots \text{XIV}$$

Losses due to heat and friction are not included. It should be noticed that this loss is dependent on n. Its amount is illustrated by the following: E changes but little with other values of P_o and p_o.

$$\begin{matrix} P_o = 100 \\ p_o = 14.7 \end{matrix} \left\{ \begin{matrix} n = 0.1 & 0.2 & 0.4 & 0.6 & 0.8 & 1.0 \\ E = 0.91 & 0.85 & 0.74 & 0.66 & 0.60 & 0.55 \end{matrix} \right.$$

Friction Losses.—In the operation of a plant the velocity in the intake pipe will be constant, but the pressure variable, while, in the discharge air-pipe, the pressure will be constant and the velocity

variable. According to Equation IX, the loss in the intake is, in pounds per square inch, Mr. Harris.

$$\frac{0.000002}{14.7}\,\frac{l}{d}\,V^2 P_x = k\, P_x = f_i \;\ldots\ldots\ldots\ldots\ldots \text{XV}$$

and the loss due to the same air passing through the discharge pipe at the pressure, P_o, is $\frac{0.000002}{14.7}\,\frac{l}{d}\left(\frac{P_x}{P_o}\,V\right)^2 P_o = k\,\frac{P^2_x}{P_o} = f_i\,\frac{1}{R_x} \ldots\ldots\ldots \text{XVI}$

To find the friction losses at intervals in the cycle, or to show such by a curve, assume convenient intervals of time (5 or 10 sec.) which indicate by t_x. Then,

$$t = \frac{Q_x}{q_a} = \frac{v\,(1+n)\,(1+k)\,\log.\frac{P_1}{P_x}}{q_i}.$$

Whence, adapting to common logarithms,

$$\log._{10} P_x = \log._{10} P_1 - \frac{t_x}{\frac{V(1+n)\,(1+k)}{q_a}}\,(0.434) \ldots\ldots \text{XVII}$$

Thus, tabulate P_x corresponding to t_x and apply the slide-rule to get the friction losses from Equations XV and XVI.

At any time, the rate of water discharge will be

$$w_x = \frac{P_x}{P_o}\,q_a.$$

This can be tabulated with the other quantities, and the friction loss in the water pipe worked out accordingly by well-known formulas. Curves worked out by the foregoing methods are shown in Fig. 7.

EDWARD WEGMANN, M. AM. SOC. C. E. (by letter).—The author states that by the old method of pumping, *viz.*, with bailing buckets, it required from 15 to 18 hours to empty the siphon under the Harlem River, while, by the plant installed by the Pneumatic Engine Company, this work could be accomplished in 12 hours. If the difference in time for pumping by the old and new methods were only from 3 to 6 hours, the wisdom of putting in the new pumping plant, at a cost of $65 827, might well be doubted, especially when it is considered that the siphon under the Harlem River may not be emptied oftener than once in ten years. Mr. Wegmann.

As a matter of fact, the old method of pumping required 30 hours or more to empty the siphon, owing to unavoidable stoppages, etc. If the new plant can do this work in 12 hours—which remains yet to be seen—the gain in time will, therefore, be 18 hours or more, and this saving in time would warrant a considerable expenditure, as the quantity of water stored at present within the limits of New York City, in the two receiving reservoirs in Central Park, only amounts to 1 180 000 gal., equal to about four days' supply.

The Aqueduct Commissioners advertised for bids for installing a pumping plant for emptying the siphon under the Harlem River. The

Mr. Wegmann. plan of the plant was left entirely to the bidder. All that was specified was that the plant should have a capacity of raising 2 500 000 gal. 337 ft. in 12 hours. On December 17th, 1901, five bids were opened by the Aqueduct Commissioners. They were as follows:

Bidder.	Amount.
The R. G. Packard Company	$55 000
Pneumatic Engineering Company	65 827
Henry R. Worthington	100 000
The Kilby Manufacturing Company	125 000
Bacon Air-Lift Company	151 500

Each bidder was required to furnish plans showing how the pumping was to be done. A brief description of the different kinds of machinery proposed may be of interest.

The R. G. Packard Company proposed to erect a plant consisting of two 30-in. single-acting lifting pumps, having a stroke of 7 ft. The water columns were to have an inner diameter of 32 in. The piston rods and connecting rods were to connect to a crank pin on balanced gearing at the top of the shaft. The pumps were to be driven by the hoisting engine which had been installed for the bailing buckets. The machinery was so arranged that the pistons, valves, pump shell and water columns could be hoisted out of the shaft, when repairs were required.

The main objection to this plan was the likelihood of the strainer in the bottom of the shaft becoming clogged, which would necessitate hoisting the piston rod, pump shell and water column out of the shaft in order to clean the strainer. What this would involve will be realized when it is stated that the piston rod was to be 5½ in. in diameter and about 400 ft. long, in lengths of 50 ft., and that the water column was to be 32 in. in inner diameter and 354 ft. long, consisting of flanged pipes, each 25 ft. long.

Other objections to the plan of the R. G. Packard Company were that there was not sufficient space in the engine-room, immediately over the shaft, for the gearing which was to operate the piston rods, and that the proposal did not include furnishing an engine for operating the pumps, as required by the specifications, but contemplated using the old engine installed for the bailing buckets.

Henry R. Worthington proposed to furnish a pumping plant consisting of a turbine centrifugal pump which was to be placed at the bottom of the shaft. The engine was to be of the cross-compound type; connected by vertical cranks to the pump shaft, and supported by heavy girders.

The Kilby Manufacturing Company proposed to furnish a pump of the six-plunger, direct-acting type. The plungers were to work horizontally, radiating from a common crank shaft, which was to be

Mr. Wegmann.

driven by direct connection with a polyphase electric motor (type "C" of Westinghouse manufacture) which was to be located directly above the pump in the bottom of the shaft. The generator, engine and switchboard were to be erected in the engine-room.

The Bacon Air-Lift Company proposed to drill four holes or submergence pits in the bottom of the shaft, each hole being 16 in. in diameter and 400 ft. deep. In each of these holes two pipes were to be placed, one within the other, and, respectively, 9⅝ and 11⅝ in. in diameter. The space between the two pipes was to be used for compressed air, and the inner pipe was to be the discharge pipe for the water, which was to enter the submergence pit in a 2-in. space outside of the air-pipe. Two valves were to be placed at the bottom of the shaft for closing the submergence pits. They were to be operated by stems extending to the top of the shaft. Two cross-compound, Corliss, two-stage air compressors were to be placed in the engine-room. Each compressor was to have a capacity equal to a displacement of 3 500 cu. ft. of free air per minute, when running at a moderate speed. The power necessary to operate the plant was estimated at from 650–700 b. h. p.

From the foregoing descriptions and that of the plant installed by the Pneumatic Engineering Company, given in the paper, it will be seen that the Aqueduct Commissioners awarded the contract for the pumping plant to the second lowest bidders, whose plan of pumping appeared to be about as practical as that proposed by any of the other bidders.

Thus far, the plant of the Pneumatic Engineering Company has not been used for emptying the siphon, nor has it ever pumped 12 hours continuously during the tests which have been made. Mr. Will I. Sando, Consulting Engineer, who conducted the tests, concluded, however, from meter measurements taken during the tests, that the pneumatic pumps had a somewhat greater capacity than the minimum required by the specifications, and that the contractors were entitled to a bonus of $1 518.48.

To keep the pneumatic pumping plant in proper condition for emptying the tunnel under the Harlem River—which may not be done oftener than once in ten years—the machinery should be run for a short time at least every month, which involves some expense. This running is also required to enable the engineer in charge of the pumping plant to become familiar with the proper manner of operating it.

Although the plant is simple in principle, it requires an exceptionally competent and cool-headed engineer to operate it. This will be evident when it is considered that the engineer has to watch a large steam-engine and two air compressors and to keep his eye on three gauges (those for the "full air pressure," the "return pressure," and the depth of water in the shaft). If the work to be done were to pump

Mr. Wegmann. from a constant level, the operation of the plant would be comparatively simple. As the conditions are at Shaft No. 25, the number of revolutions of the fly-wheel has to be varied according to the depth of the water in the shaft, and the auxiliary compressor has to be put in operation whenever it has to make good a loss of air. Should the engineer have too much air pressure, one of those "blow-outs" described by Mr. Diamant would occur, and might cause serious damage.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 989.

PROBABLE WIND PRESSURE INVOLVED IN THE WRECK OF THE HIGH BRIDGE OVER THE MISSISSIPPI RIVER, ON SMITH AVENUE, ST. PAUL, MINN., AUGUST 20TH, 1904.*

By C. A. P. Turner, M. Am. Soc. C. E.

With Discussion by Messrs. Theodore Cooper, George E. Gifford, L. J. Le Conte, Charles L. Strobel, E. P. Goodrich and C. P. Turner.

In view of the fact that the wreck of a well-braced iron or steel structure by wind is exceedingly rare, if, indeed, there is any previous record of such, the destruction of part of the so-called High Bridge over the Mississippi River at Smith Avenue, St. Paul, would seem to be of special interest to the professional bridge engineer.

This structure, Plate IV, was designed supposedly to meet, with a reasonable factor of safety, the maximum wind loads required by a standard specification; and the utter destruction of a portion of it by wind pressure alone, in view of this supposed margin of safety, would lead to the presumption that the standard requirements do not produce a safe structure, unless it can be shown by reasonable computation that there was some weak joint or detail in the frame which

* Presented at the Meeting of December 7th, 1904.

would insure its destruction under the action of forces not materially greater than those which, nominally, it was designed to withstand.

The structure was built in 1887, according to general plans prepared by the City Engineering Department of St. Paul. Detailed drawings were made by the Contractor, C. L. Strobel, M. Am. Soc. C. E., and the work was erected by Horace E. Horton, M. Am. Soc. C. E., of Chicago, Ill.

The bridge is a deck structure of wrought iron, 2 770 ft. long, and runs northwest and southeast. The northwest portion of the bridge is of the viaduct type, with riveted spans of 80 ft. and plate-girder tower spans of 40 ft. Four-leg towers alternate with two-leg bents. The portion of the viaduct over the river consists of four 250-ft. pin-connected deck spans of the subdivided Warren type, 30 ft. deep and 22 ft. from center to center of trusses. The floor beams are at 12 ft. 6 in. centers. The tower supporting the shore end of the southeast 250-ft. span has a base of 55 ft. transversely, and of 50 ft. longitudinally, and a height of 129 ft. from the top of the pier to the bottom chord of the truss. As these trusses were 30 ft. deep, the roadway at this point was 160 ft. above the pier and about 180 ft. above the water. From this tower toward the bluff there was one 170-ft. pin span and two 60-ft. plate-girder spans.

These girder spans, the 170-ft. pin span, the supporting tower, and the 250-ft. pin span were overthrown, as shown in Plates V, VI and VII.

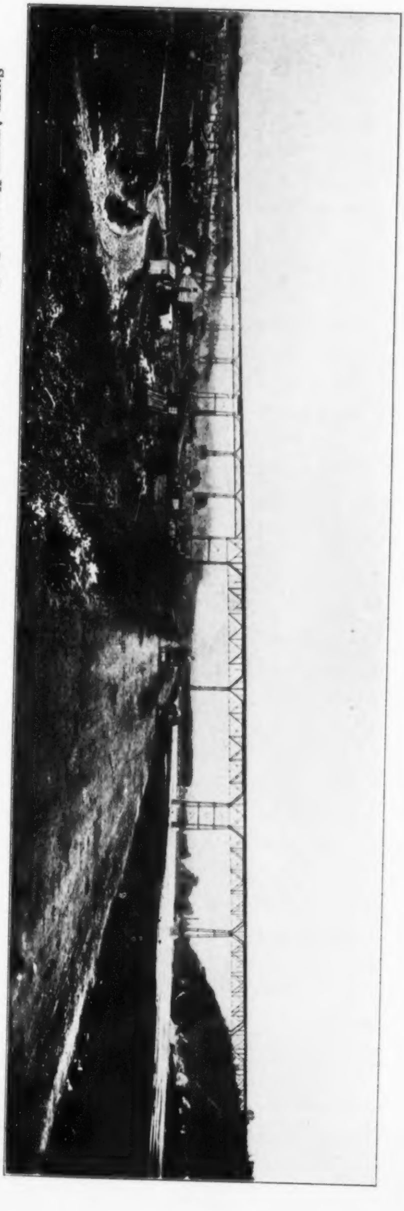
The bridge carries a 25 ft. roadway and two 8-ft. walks. The flooring for the roadway consists of a sub-floor of $3\frac{3}{4}$-in. fir plank and a wearing floor $1\frac{3}{4}$ in. thick. The plank for the walks is $2\frac{3}{4}$-in. pine. The stringers are of steel, the roadway of nine lines of 12-in. built stringers; the flanges are each two L's, 2 by 2 by $\frac{5}{16}$-in., with $\frac{5}{16}$-in. webs. The stringers for the walks are 6-in. I's.

The trusses were designed for a live load of 80 lb. for the roadway and for the walks of the 250-ft. spans, 90 lb. for the 170-ft. span and 100 lb. per sq. ft. for all shorter spans.

The lateral bracing was designed for a pressure of 450 lb. per lin. ft. of bridge, two-thirds of which was assumed to act on the loaded (upper) chord. The towers and bents were assumed to have a wind pressure of 150 lb. per lin. ft. acting against them.

In addition to the top and bottom lateral systems, a fairly efficient system of sway rods was provided in the 250-ft. span, and all the de-

PLATE IV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 989.
TURNER ON
WIND PRESSURE ON BRIDGES.



SMITH AVENUE VIADUCT, ST. PAUL, MINN., OVER MISSISSIPPI RIVER, VIEW FROM THE UP-STREAM SIDE, BEFORE THE STORM OF AUGUST 20TH, 1904.

tails of the lateral and sway bracing seem to have been well worked out for the type of bracing used.

Referring to Figs. 1 and 2, Plate V, it will be noted that the 250-ft. span is lying on its side, except at the end torn from its support on the two-leg bent (still standing), and that this end has been given a quarter twist in addition and has fallen or has been blown a considerable distance from the tower.

Referring to Figs. 1 and 2, Plate VII, the plate-girder spans seem to have been pulled down the bank, and are but little out of the line of the viaduct.

The tower frame which was overthrown was badly twisted, and the position in which the columns fell, together with the manner in which the bolts were bent and broken, would seem to indicate that the end of the 250-ft. span resting on the two-leg bent was first pushed off its support, and that the wind, acting on the loose span with its 10 000 or 11 000 sq. ft. of exposed area (the planking was well fastened) and an extreme leverage of 250 ft., twisted from its base the tower bent supporting the other end, and the falling mass, in its descent, pulled the girder spans down the hill.

If the collapse occurred as outlined, a few figures on the twisting moment on the top of the tower may be in order. Supposing the floor to be at such an angle to the wind that the effective pressure is, say, 10 lb. per sq. ft., then the twisting moment $= 10 \times 10\,000 \times 125$ ft. $=$ 12 500 000 ft-lb., an amount far in excess of the ultimate resistance of the tower.

The next point which would seem to invite attention is the detail of the connection of the end of the wrecked 250-ft. span to the two-leg bent, and the strength, or resistance of this connection to uplift and to lateral sliding of the shoe.

Referring to Figs. 1 and 2, Plate V, it will be noted that there is a two-leg bent, similar to the one that supported the end of the wrecked span, nearer the other shore, and, as this seat was easily accessible by a trap in the floor, a suspended platform and an iron ladder leading down to the shoe, it was examined first. The end of the 250-ft. span corresponding to that which was wrecked was found to rest on a nest of eight rollers, each about $2\frac{3}{4}$ in. in diameter, with the usual spacing bars on the sides.

The sole resistance to the lateral motion of the rollers was a bar riveted to the cap on each side and a recess in the shoe above, about $\frac{1}{8}$

or $\frac{5}{8}$ in. in depth, as nearly as could be readily determined. Provision was made for a $1\frac{1}{2}$-in. guard bolt on each side of the shoe, a hole was provided in the column cap and a long slot in the side of the shoe. No bolts, however, were in place. The photograph, Fig. 2, Plate VIII, taken from the platform vertically above the shoe, shows clearly the hole where this bolt should have been, on the outside of the shoe, on the up-stream side of the bridge; and Fig. 1, Plate VIII, shows the absence of the bolt on the down-stream side. The inner sides of the shoes could not be photographed conveniently, but the bolts were missing there also.

Fig. 2, Plate IX, is a photograph of the column cap from which the 250-ft. span slid off on the leeward side of the bridge, and Fig. 1, Plate IX, is a view of the cap on the windward side. Each of these views was taken looking diagonally downward from the end of the floor still standing. It may be noted that but one roller has been left on the windward cap while seven of the eight remain on the leeward cap. Careful examination of these photographs will show that this span was anchored down somewhat better than the one referred to above, and instead of having no bolts at all, there was one on the outside of the windward shoe which is splintered and broken in place. The appearance of the other three holes is positive evidence that there were no bolts in any of them.

There is, then, the resistance of this end of the span, reduced to the dead weight, and the value of this bolt. If the wind tended to raise the windward truss, as it is pin-connected with the eye-bar bottom chord and diagonals, the truss would furnish little resistance to upward forces, and the bolt at the end, being a cantilever from $3\frac{1}{2}$ to 4 in. from the center of the shoe plate to the center of the bearing in the cap, would not develop its shear value, but only its bending value, the insignificant amount of 3 000 or 4 000 lb. or less.

An uplift on the windward side would be accompanied by a reversal of stress in the bottom chord, the probable buckling of the chord, and, with the slight resistance of the bolt, the shoe would be pulled from the cap and the rollers displaced, as appears in Fig. 1, Plate IX.

A rough approximate estimate of the weight of the span and floor would be in the neighborhood of 2 200 lb. per lin. ft., giving a reaction of, roughly, 140 000 lb. at each support. As the storm was a severe one, it will be assumed, for purposes of computaion, that the

FIG. 1.—SMITH AVE. VIADUCT. VIEW LOOKING NORTHWEST. WRECKED BY STORM OF AUGUST 20TH, 1904.

FIG. 2.—SMITH AVE. VIADUCT. VIEW LOOKING NORTHWEST.

very severe wind pressure of 30 lb. per sq. ft. was acting at an upward angle to the floor of 30°, and the pressures will be calculated in accord with Unwin's table.

Let a = Angle of surface with direction of wind;

F = Force of wind, in pounds per square foot (assumed at 30 lb. per sq. ft.);

A = Pressure normal to surface $= F \sin. a^{1.84 \cos. a - 1}$;

C = Pressure parallel to direction of wind $= F \sin. a^{1.84 \cos. a}$.

For $a = 30°$, $A = 0.66$ and $C = 0.33$.

The direct uplift at each shoe $= 20.5 \times 125 \times 0.66 \times 30$ lb. $=$ 50 100 lb. from the wind on the floor.

The overturning force, C, at each end of the bridge $= 0.33 \times 30$ lb. $\times 41 \times 125 = 51\,200$ lb. from the wind on the floor.

The uplift from C on the windward shoe $= \dfrac{51\,200 \times 30}{22} = 70\,000$ lb.

The direct pressure on the side of the truss top chord, approximately, $= 30 \times 7 = 210$ lb. per lin. ft., and $\dfrac{210 \times 125 \times 30}{22} = 36\,000$ lb., the uplift from the same.

As the wind has been assumed to be blowing upward, this component on the vertical area would give an additional uplift of some 6 000 lb.

Now, the sum of these computed uplifts is 162 100 lb., or about 15% greater than the reaction due to weight.

Evidently, if the windward shoe is raised, there being no bolts to hold down the leeward shoe, it would turn sufficiently for the recess in the shoe to clear the corner of the rolls and then slide off the cap.

Allowing some slight resistance for the expansion connection of the stringers to the beam, it would seem safe to conclude that the wind pressure assumed is 10% greater than would have been necessary to cause the wreck.

Rough computations on the laterals, taking into consideration the sway rods and the action of the four planes of bracing, would indicate that they were not strained much beyond 23 000 to 25 000 lb. under the assumed forces.

Bearing in mind the fact that the floor is on a steep up grade, it may well be that the angle of action of the wind on the floor was greater than has been assumed, and, if so, the necessary pressure to cause the wreck might be considerably less than the 27 lb. per sq. ft.

calculated. Again, the probability is that the assumption of a uniform pressure is materially in error. Judging somewhat by the contour of the bluff and the path of the storm, it would seem likely that the maximum pressure was in the vicinity of the northwest end of the wrecked 250-ft. span, and that the adjoining span was saved by its rigid connection to the two-leg rocker bent. If the pressure were greater at the end, it is evident that the average pressure necessary to cause the wreck would be materially less.

Such moderate pressures as have been figured on, when their cumulated effect is concentrated upon a weak detail, may evidently produce results that cause astonishment, and the rash assumption, by those whose training should lend better judgment, that the pressures involved are "exceedingly great."

Evidently, whether dealing with bridges or roofs, stiff riveted construction, with bottom chords and diagonals capable of taking reverse stresses, is to be preferred, and, in view of the fact that, by the exercise of reasonable skill in design, they can be fabricated for a sufficiently smaller cost than the pin type to offset the additional expense of riveting in the field, they should be preferred for all moderate spans, such as 250 ft. or less, unless the work is exceptionally heavy.

In the provision for temperature stress, the expensive and frequently weak details often worked out to avoid a harmless little amount of temperature strain, in an effort to eliminate it entirely, is indeed surprising; perfect double-action joggle connections are too often introduced at the shortest possible intervals, and dignified by the name of expansion joints.

In the present instance, for example, an ordinary sliding plate fitted with a compression grease cup would probably move as easily as the badly rusted rolls on a rusty base and cap; while the guard bolts would be brought into actual shear and tension under forces tending to displace the shoe instead of inbending as with the detail adopted.

From a careful examination of the 2½-in. anchor bolts of the windward column of the fixed bent under the 250-ft. span, it would seem that they were without nuts, though this fact appears to have had no material influence on the wreck.

FIG. 1.—SMITH AVE. VIADUCT. VIEW LOOKING SOUTHWEST. SHOWING PART OF SHORE TOWER, AND ALSO RAILROAD TRESTLE CUT THROUGH BY THE FALLING TOWER.

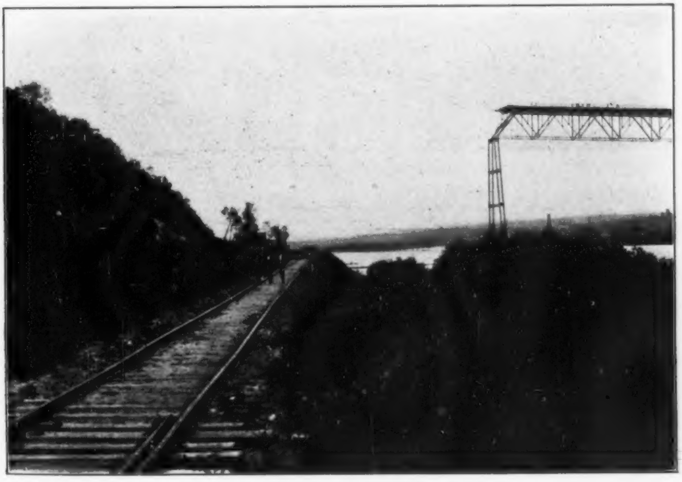


FIG. 2.—SMITH AVE. VIADUCT. LOOKING UP STREAM. SHOWING SECTION OF VIADUCT THROWN DOWN BY STORM.

DISCUSSION.

THEODORE COOPER, M. AM. SOC. C. E. (by letter.)—Mr. Turner deserves the thanks of the Profession for his investigation of the wreck of the St. Paul bridge. His explanation of the failure appears to be the probable one. For many years the writer has watched the reports of tornadoes and their effects, and has yet to find a case of a properly designed and constructed bridge which has been destroyed by the wind. Mr. Cooper.

While many bridges, roofs and other structures have been overturned or destroyed by the wind, the writer has not found one case which indicated that the modern requirements for wind bracing have proved inefficient. In numerous cases, the wind has been made the cloak to cover the ignorance, inexperience, neglect or chicanery of the designer or builder. The wind has been very much maligned, and, in the writer's opinion, its power has been very much over-estimated.

While preparing the Erie Specifications, in 1879, the writer found a memorandum, by the late George S. Morison, Past-President, Am. Soc. C. E., formerly Principal Assistant Engineer, giving the sizes of lateral rods in such spans as were acting stiffly under the trains. Using this as a guide, and also finding that it corresponded very closely with the results of 30 lb. per sq. ft. of exposed surface, a lateral force per linear foot of span was adopted for "wind and vibration."

A few years ago it was brought to the writer's attention that, under modern train loads and speeds, this rule did not give sufficient rigidity, and in his later specifications it has been increased one-third.

This amount of lateral stiffness, therefore, is needed in bridges of ordinary lengths of span, regardless of the wind, and, as it has proved satisfactory under the wind forces also, there is no reason for changing it, except it be found desirable to give increased stiffness under moving loads.

All authorities agree that an absolutely steady wind is unknown; that the wind is a series of eddies and gusts; that the records of anemometers are only the measures of the highest gusts upon small areas and only represent an instantaneous and limited effect. It follows naturally that on larger areas the average pressure at any moment will be less than that recorded by the pressure gauge. At the Forth Bridge the pressure on a surface of 20 by 15 ft. was practically only about two-thirds of that shown on a gauge having an area of 1½ sq. ft. For larger spaces the reduction would probably be still more marked. The irregular waves of a field of grain acted upon by the wind make clear to an observer the effects of the gusts and why on large spaces the full effect is never possible at any one instant.

Since Smeaton's time a wind of 100 miles per hour has been rated

Mr. Cooper. as one that would uproot trees and move buildings. Such a wind, according to accepted formulas, would exert a maximum pressure of 40 lb. per sq. ft. on small areas. A pressure of 25 lb. per sq. ft. acting at the same moment on large areas would, in the writer's opinion, denude a district of trees and buildings, and it is very unlikely that a wind of 100 miles' velocity exerts a force greater than this on objects of ordinary size.

General Greeley, of the United States Signal Service, in 1890, after the Louisville, Ky., tornado, stated:

"As bearing upon the strength of structures necessary to withstand tornadic winds, it is important to note that there have been very few cases recorded of wind velocities in the United States where the pressure of the wind, according to the latest investigations and accepted formulæ, exceeded 16 lb. to the square foot."

The severest effects of the wind occur in the paths of tornadoes. These, however, are very limited in their breadths, the high pressures or destructive effects being limited to a few hundred feet, and then the results are usually recognized as being due to the oscillation of the path of a single vortex or of a series of vortices following one another. It is probable that the destructive action of a single vortex does not exceed a breadth of about 60 ft., as has been frequently observed. Moreover, being a rotating force, it could not exert its pressure in any one direction for more than half its breadth.

The estimated pressures or velocities to produce the greatest recorded results, such as lifting locomotives, breaking off the top of an obelisk, twisting iron bars, driving straws through pine boards, etc., amount to only about 150 lb. per sq. ft., or 200 miles per hour.

Julius Baier, M. Am. Soc. C. E., in his excellent paper on the last St. Louis tornado,* found evidences of pressures as high as 60 lb. over a length of 180 ft.

At the St. Charles Bridge, the late C. Shaler Smith, M. Am. Soc. C. E., reported a tornado which exerted a pressure of 52 lb. on small objects on the bridge and 84 lb. in the vicinity, and which did not injure the bridge, although its bracing was only proportioned for 30 lb. per sq. ft. of exposed surface. He also found, after following up the paths of several tornadoes, but one case where 60 ft. of width was not enough to cover the path within which the computed pressures exceeded 30 lb.

Many careful investigators of the effects of high winds and tornadoes have concluded that it is very improbable that they ever exert an effect of 30 lb. per sq. ft. over a space of 150 to 200 ft., at one time.

When one considers, then, the lateral force for which bridges have been designed, and the limiting strains allowed for all bracing, one has a right to expect that any properly designed and constructed bridge

* "Wind Pressure in the St. Louis Tornado, with Special Reference to the Necessity of Wind Bracing for High Buildings." *Transactions*, Am. Soc. C. E., Vol. XXXVII, p. 221.

FIG. 1.—SMITH AVE. VIADUCT. VIEW LOOKING NORTHEAST, SHOWING PARTS OF SHORE TOWER, 170-FT. SHORE SPANS AND TWO 60-FT. PLATE GIRDERS.

FIG. 2.—SMITH AVE. VIADUCT. VIEW FROM END OF VIADUCT STILL STANDING, SHOWING PART OF WRECK AND ABUTMENT OF WRECKED END.

Mr. Cooper.

will escape injurious effects from high winds or tornadoes. However strong the bridges may be braced, if they are not properly anchored down or stayed against being shoved off their seats—two very common faults—they may at some time fail.

Regardless of the very strong probability that a lateral force equal to the ordinary requirements of our specifications is never exerted by the highest winds on spans of more than 150 ft., and not oftener than once in a generation on shorter spans, the bracing due to such requirements could not be reduced without rendering these bridges inefficient against the vibrations of moving loads. But an era of very long spans has been entered, in which it is desirable and necessary to take a broader view of this subject.

To design a 2 000 to 3 000-ft. span, which the writer believes will not be an uncommon length for the next generation of bridge builders, and may even occur as a case for some of the present generation, under the same wind requirements as for spans of 500 ft. and less, would be very unscientific and wasteful.

The development of these longer spans and the limitation of the greatest possible span depend very largely upon the assumed wind force. The wind force, being practically a horizontal force, while the dead and live loads are vertical forces, any unnecessary material added for impossible wind forces is a detriment to the structure and a handicap against progress.

Sir Benjamin Baker gives the following resultant stresses per square inch, on the top and bottom members of the Forth Bridge, for dead, live and wind forces, in tons.

	Dead load.	Live load.	Wind.	Total.
Top member..........	4.4	2	1.1	7.5
Bottom member.......	2.8	1.2	3.5	7.5

Bearing in mind that an important part of the dead load was due to the material added for the wind stresses, it will be seen that in the lower members the wind exerted a greater influence than the dead and live loads together.

The Board of Trade required the Forth Bridge to be built to resist a "wind force of 56 lb. per sq. ft., striking the whole or any part of the bridge at any angle upon an area equivalent to twice the plane surface of the front girders, with a reduction of 50% in case of tubes."

Fifty-six pounds per square foot of surface striking simultaneously a length of 1 700 ft., when the highest possible claim that could reasonably be made from the evidences of the worst known storms would not give this pressure over 200 ft.!

It may be unnecessary to say that this requirement was imposed upon the eminent engineers of this bridge, and was not the result of their own conclusions.

Mr. Cooper. It might be possible that a wind force of 56 lb. per sq. ft., covering a lateral extension of 1 700 ft., through its oscillatory movements, could occur, but, from all the evidence of the character and action of such high winds, it is impossible to conceive of such a wind striking a simultaneous blow of this force over 100 ft. of lateral distance.

The absurd requirement forced upon the Forth Bridge should not be accepted as a precedent. It is time that a more rational requirement for long spans should be attempted.

For spans up to about 500 ft. the existing requirement of a fixed lateral force per foot of span should not be relaxed, as it is desirable to have this much rigidity against the action of moving loads.

For very long spans, where the exposed surfaces become much larger, the lateral force sufficient to give rigidity under moving loads may not be enough to provide for possible wind forces.

For all spans exceeding 500 ft. in length the writer would suggest the following wind forces as sufficient to cover all cases.

First.—A wind force of 50 lb. per sq. ft. acting at the same moment over a width of 60 ft., striking any part of the bridge at any angle within 30% above or below the horizontal;

Second.—Similarly, a wind force of 30 lb. over a width of 600 ft.;

Third.—Similarly, a wind force of 15 lb. over a width of 2 000 ft.; the maximum stresses from either of these requirements to be used for proportioning each member.

As all these requirements are far beyond what recorded evidence would lead us to believe as probable; as their duration, should they occur, is for a very short time, acting on a mass of great inertia; and as their recurrence would only be at long intervals of time, engineers would be justified in using high unit strains for the combined dead, live and wind loads. Assuming that for very long spans two-thirds of the elastic limit of the material for dead and live loads combined is not exceeded, they could use safely $33\frac{1}{3}\%$ more, or eight-ninths of the elastic limit for the dead, live and wind strains combined, for the truss members.

Though the foregoing wind requirements, in the writer's opinion, are excessive, they are so much more reasonable than the usual ones specified for long-span bridge projects that it is desirable to have them discussed.

Their acceptance, after such modifications as may be developed by discussion, would vastly improve the possibilities of future long-span bridges.

Mr. Gifford. GEORGE E. GIFFORD, M. AM. SOC. C. E.—The speaker believes that the probable cause of the disaster which forms the subject of Mr. Turner's paper is correctly stated by the author, and in this also agrees with Mr. Cooper.

There is one exception which might be taken to the conclusions

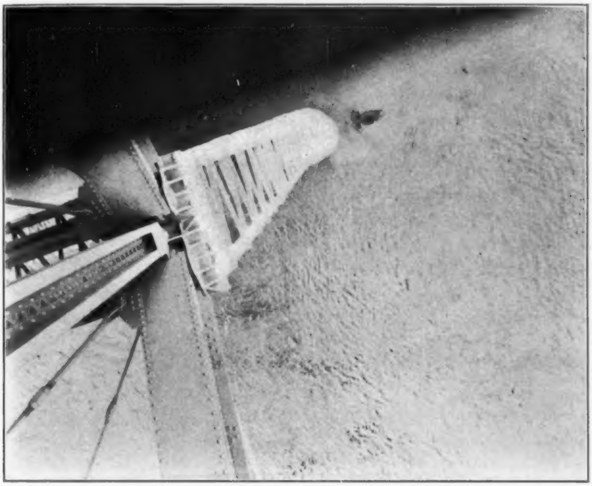


FIG. 1.—SMITH AVE. VIADUCT. VIEW LOOKING VERTICALLY DOWN AT EXPANSION AND FIXED SHOES OF 250-FT. SPANS ON ROCKER BENT, SHOWING ABSENCE OF BOLT ON DOWN-STREAM SIDE.

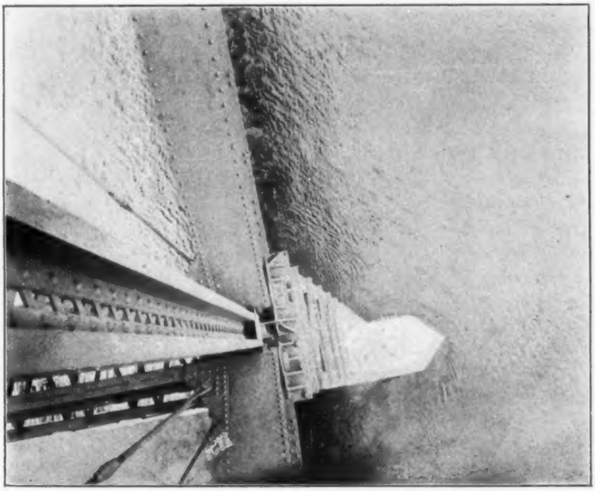


FIG. 2.—SMITH AVE. VIADUCT. VIEW LOOKING VERTICALLY DOWN AT EXPANSION AND FIXED SHOES OF 250-FT. SPANS ON ROCKER BENT, SHOWING ABSENCE OF BOLT ON UP-STREAM SIDE.

drawn by the author: His statement, or rather, intimation, that a riveted structure with stiff members throughout would have been preferable, does not seem to be borne out by the facts. It is not clear that the same thing would not have happened, whatever the type of structure, since the failure seems to have been caused by deficient bolting to supports, or lack of dead weight. The dead weight might have been greater, and probably would have been, had it been a riveted structure, but the author does not take this into account. Nor would it have been sufficient to hold the truss down, under the conditions stated, in the absence of proper bolting. Mr. Gifford.

The speaker does not at this time intend to discuss the merits of riveted *versus* pin-connected trusses, but it is questionable whether a riveted truss is preferable, all things considered, for a highway span of 250 ft. It certainly cannot be fabricated in the shop at a sufficiently lower cost to offset the additional material required and extra cost of erection.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The facts relating to the failure of this viaduct to withstand wind pressures are extremely interesting, and such cases are always instructive. It is hardly to be expected that, at this late date, any defective detail will be definitely discovered and recorded. All that can be done, therefore, is to study probabilities. In the absence of such information, it is but fair to presume that the viaduct was constructed in every way up to the requirements of the specifications of that date, 1886. Mr. Le Conte.

Judging from the photographs of the wreck, however, Figs. 1 and 2, Plate V, and also Fig. 1, Plate VI, the writer is rather inclined to think that in all probability the maximum wind effect was at or near the site of the four-leg tower, at the southeast end of the 250-ft. span. Everything in the photographs seems to point that way. For example: Fig. 2, Plate VII, showing the four pedestals and the wrecked tower, seems to indicate an ordinary case of "tip over," pure and simple, due most probably to lack of proper anchorages to the masonry pedestals. This is indicated by the direction of the upper chord of the 250-ft. fallen span, as it lies in the river. It does not occupy a position which would indicate that the northwest end of the 250-ft. span had given way first. On the contrary, the photographs seem to show that the distance of the upper chord from the original center line of the bridge, at the four-leg tower base, is certainly much greater than the corresponding distance of the upper chord, at the northwest end of the fallen span, from the center line; thus showing, apparently, a decidedly angular position, which, prolonged, cuts into the alignment of the bridge proper very perceptibly. It appears to be clear, therefore, that the 250-ft. span was undoubtedly thrown farthest out at the four-leg

Mr. Le Conte. tower site, and, consequently, this was most probably the locus of the maximum wind effect upon the bridge. It seems to the writer, therefore, more probable that the four-leg tower was torn from its four-pedestal base completely and, toppling over, pulled the other spans after it and the 250-ft. span off its seat on the two-leg bent.

Moreover, it should be remembered that the specifications in vogue in 1886, when the bridge was designed, were probably defective as to anchorage. At least, the writer knows that the everyday practice of that date was the plain, old, ordinary connection of columns to pedestals by an anchor bolt at each of the four corners of the bed-plate. This constituted a miserably weak and ineffective detail for such a high tower. Modern specifications demand that the anchorage shall hold the foot of the column to the pedestal so securely that failure by overturning or rupture at the footing could not occur if the bent were tested to complete destruction, whereas, Fig. 2, Plate VII, shows plainly that the four pedestals, after the wreck, were left perfectly clean and free from wreckage, apparently as though there were no anchor bolts in existence.

Just one word more on wind pressures in general: Past records as to wind pressures are extremely vague and unsatisfactory. Of late years, however, instruments have been devised by which the pressures are recorded with commendable accuracy. The results obtained are startling when compared with popular estimates made at the same time. This discrepancy, in a measure, explains itself. The intensity of the wind pressure is generally confined to very narrow limits, and, as a result, two pressure gauges do not give anywhere near the same record; simply because the pressure was not there to be recorded. Hence, no single observation of pressure is of much value, simply on account of the well-known and great variation in pressures within short distances. What the bridge engineer really wants to obtain, most of all, is a reliable average for a given entire span; and, as might be naturally inferred, within limits, the longer the span the less the average.

The writer does not think that a rational conclusion can possibly be arrived at, except by actual observations on existing bridges. This information is now very badly needed, more particularly when making specifications for long-span bridges.

Mr. Strobel. Charles L. Strobel, M. Am. Soc. C. E. (by letter).—The author assumes a wind acting upward at an angle of 30° to the horizontal and blowing against the underside of the floor of the bridge with a pressure of 30 lb. per sq. ft. in that direction. This floor has a width of 41 ft., and, on this assumption, the exposed area is therefore very considerable. This force he divides into two components, one acting normal to, and the other parallel with, the floor. The force normal to the floor is assumed at 20 lb. (in round figures) per

FIG. 1.—SMITH AVE. VIADUCT. VIEW LOOKING DIAGONALLY DOWNWARD ON WINDWARD COLUMN CAP, FROM WHICH THE CORNER OF THE 250-FT. SPAN WAS LIFTED, AND THE BROKEN REMNANT OF THE ONLY BOLT HOLDING IT IN PLACE.

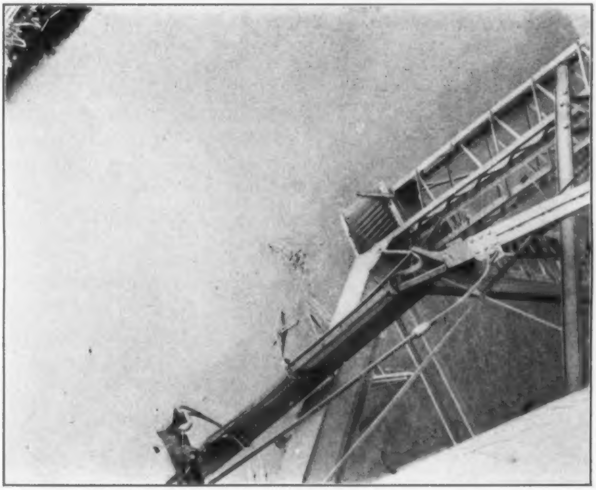


FIG. 2.—SMITH AVE. VIADUCT. VIEW LOOKING DIAGONALLY DOWNWARD ON LEEWARD COLUMN CAP FROM WHICH THE SHOE OF THE 250-FT. SPAN SLID OFF.

Mr. Strobel.

sq. ft. and the horizontal force at 10 lb. per sq. ft. These figures were obtained from Unwin's table for wind pressures on roofs, but the writer thinks that the use made by the author of the latter figure is erroneous. In that table this figure is the horizontal component of the normal pressure, which, there, is a force inclined to the vertical. On this account, it is convenient to make use of the horizontal and vertical components of the normal pressure instead of utilizing the latter, direct.

In this case, however, the normal force is vertical, and is used by the author in full; therefore, a component of this force should not also be used. The horizontal component of the original inclined force of 30 lb. per sq. ft. against the underside of the floor, on the other hand, exerts a pressure upon the bridge only through friction of the air against the floor, and this friction would be only a very small percentage of this force, so small that it is usual to neglect it. The writer thinks that the horizontal force of 10 lb. against the underside of the floor should be eliminated entirely from the author's calculations. Making this correction, and increasing the pressure against the sides of the trusses from 210 lb. (= 30 lb. $\times$ 7) per lin. ft., assumed by the author, to 300 lb., the specification requirement, we obtain, for the 250-ft. span, the dead load required for equilibrium

$$= \left\{ 300 \text{ lb.} \times 125 \times \frac{30}{22} \text{ (overturning force)} + 50\,100 \text{ lb. (uplifting force)} \right\} \times \frac{2}{125} = 1\,620 \text{ lb.}$$

per lin. ft. of bridge. In other words, the 250-ft. span will not be overturned by a horizontal wind pressure of 300 lb. per lin. ft. acting in the plane of the top chords, combined with a normal upward pressure of 20 lb. per sq. ft. against the floor of the bridge, if the dead weight exceeds 1 620 lb. per ft. Even taking the dead weight at only 2 200 lb., as roughly estimated by the author, it is clear that the destruction of the bridge could not have been caused by the overturning of the 250-ft. span, which was the author's conclusion, unless the wind pressures were much greater than he assumes.

Upon inquiry, the writer finds that the wooden floor of the bridge is different now from what it was when the bridge was originally built, sixteen years ago. The upper or wearing floor was 4-in. cedar blocks then; it is 2-in. oak plank now. The dead weight of the 250-ft. span is given at 3 200 lb. per lin. ft. on the original stress sheet, as against 2 200 lb. assumed by the author, which is evidently much too small, even for the present floor.

The specifications, under which the bridge was built, provided for horizontal wind pressures only, and these were 300 lb. per lin. ft. of bridge in the plane of the top chords, and 150 lb. per lin. ft. of

Mr. Strobel. bridge in the plane of the bottom chords. These are Cooper's requirements for wind for highway bridges. They were standard then, and are standard to-day. For these wind pressures, anchor bolts are required to resist overturning, neither at the end of the 250-ft. span nor at the foot of the tower. At the end of the 250-ft. span the dead load exceeds the overturning force of the wind by 148 900 lb. for each shoe, and at the foot of the tower, for the bent supporting the 250-ft. span, by 68 300 lb., according to the original stress sheet. Assuming the author's lighter dead weight, these figures would be 86 400 lb. and 5 800 lb., respectively, but his dead load is probably 840 lb. per ft. too small for the present floor. The bridge, therefore, would not have failed under the wind pressures allowed for, even if the anchor bolts were without nuts, the condition in which some of them were found. Although, as shown, under the original dead loads, anchor bolts were not needed to hold down the bridge, they were provided as an extra precaution, and were of liberal size. They were intended primarily to hold the bed plates in position laterally, and were run through the bottom plate of the shoe, but not with the desire of developing the full strength of the bolts in tension. To do this it would have been necessary to increase the thickness of the bottom plates. Still, these anchor bolts were good for a considerable resistance to an uplifting force, and, of course, they should have been kept in proper working order.

This bridge was designed with the same care and attention to details as any modern railroad bridge. Shop plans were submitted for approval, and were carefully gone over by a thoroughly competent, conscientious and painstaking engineer, A. W. Münster, M. Am. Soc. C. E., then Bridge Engineer of the City of St. Paul.

The author's theory, that the bridge failed by the overturning of the end of the 250-ft. span, which rested on the rocker bent, seems improbable from the fact that this would require so large an upward pressure against the wooden floor that the clinched wire nails, with which the floor was fastened to the iron stringers, would be torn off before the bridge could be lifted up.

A much more probable theory would appear to be that the bridge was wrecked by the overturning of the tower at the shore end of the 250-ft. span, the other spans following in natural sequence. At the tower the surplus of dead load to resist overturning was very much less than at the end of the 250-ft. span, as has been shown. Furthermore, beginning at or close by this tower, the shore rises abruptly on the south side of the river. The wind is reported to have blown diagonally against this slope, and therefore would have been deflected upward so as to exert considerable pressure against the underside of the floor. This pressure was probably sufficient to

Mr. Strobel.

overturn the tower, especially if the anchor bolts were partly without nuts, as reported.*

As to the lessons taught by the failure of the bridge: the first and most obvious one would seem to be that anchor bolts for high bridges or viaducts should be looked after, and nuts replaced if lost or stolen; secondly, that the floor of a high viaduct should not be made lighter in weight without an investigation as to whether the anchorage provided is still sufficient to resist wind, and, if not, to strengthen the anchorage as may be required by the new conditions.

The writer's conclusion, in general, is that our present assumptions for wind pressure did not prove adequate to save this bridge, and that either the intensity of the wind was much greater than the pressures ordinarily allowed for, or that the wind, though not of greater intensity, was deflected upward by the steep slope of the high south shore and thereby given an intensified effect sufficient to cause the failure of the spans near this end, under the unfavorable conditions of a lightened floor and anchor bolts apparently in bad order.

The writer agrees with Mr. Cooper that, in the future, provision should be made for certain upward wind forces, but, in any general specifications, these forces should be stated definitely, and not left to be deduced from wind pressures acting at a certain angle with the horizontal.

The old method of obtaining the wind pressures, by the parallelogram of forces, is obsolete, as it has been shown by modern experiments that the normal pressures can be deduced from experiment only. A splendid beginning, in the scientific investigation of wind pressure, has been made by the Committee of the British National Physical Laboratory,† and further experiments are promised, so that we may soon hope to know more about wind pressure than heretofore. Attention is called to the fine opportunities for observation on a large scale afforded by modern bascule bridges, and those in a position to do so are urged to make and publish records of the power required to operate these bridges in high winds, measuring at the same time the direction and velocity of the wind. Such observations would be very valuable.

Mr. Goodrich.

E. P. Goodrich, Jun. Am. Soc. C. E. (by letter).—The subject of wind pressures has interested the writer for several years past, during which period he has been making continued observations and investigations, both mechanical and analytical. He also had some very vivid experiences in the cyclone which struck Ypsilanti, Mich., on the evening of April 12th, 1893.

---

* See Mr. R. A. Tanner's account of the failure, showing the position of the wrecked structure, *Engineering News*, December 8th, 1904.

† See Stanton on "The Resistance of Plane Surfaces in a Uniform Current of Air," *Minutes of Proceedings*, Inst. C. E., Vol. CLVI (June, 1904).

Mr. Goodrich. While the damage done to the St. Paul bridge is well described by Mr. Turner, his description of the storm which wrecked it is rather meager. From technical publications, from the United States Weather Bureau, and by letter from Mr. Turner, personally, the writer has gathered a few additional facts. The storm was cyclonic; came against the bridge almost normally; had a width of path of destructive effect of approximately 250 ft.; developed wind velocities of 110 and 125 miles per hour, at the Weather Bureau observatories in Minneapolis and St. Paul, respectively; and caused a variation in barometric pressure from 28.40 to 28.25 and then to 28.80 in.

In his observations of the storm at Ypsilanti, the writer noted many facts which were similarly noted by Mr. Turner, as published at the time of the St. Paul storm.* It is the writer's opinion that, in such storms, the most damage is done by the rush of air toward and up the vortex which forms the storm center. In several instances in Ypsilanti the brick walls of well built houses were burst outward on two sides while the edge of the track of the vortex seemed to be several feet distant. Mr. Turner† describes a roof which was burst outward on the leeward side of the gable while the windward side remained in position. Many such published and personally made observations have convinced the writer that there is apt to be as much suction on the lee side of thin structures, like walls, bridge floors, etc., and on the lee side of many gable roofs, as pressure upon the windward side. He has always made it a practice to take due account of these facts when designing the supports of roof trusses, etc. It seems as if the actual cause of the accident described by Mr. Turner was a failure to appreciate and provide for this condition.

The actual pressures due to non-cyclonic winds are of equal importance to the engineer, and should be as carefully studied. Mr. Cooper has proposed a most sensible specification, in the writer's opinion, in requiring designs to withstand pressures of decreasing amount, as greater areas are considered. The observations made by the writer, while not yet complete enough for publication, and the analytical investigations which followed the securing of the first consistent data, seem to point to a wind pressure which varies inversely as the logarithm of the least dimension of the area in question. The law of the variation of the pressure with the square of the velocity also holds for any given area through the usual range of variation, but many formulas used for the reduction of anemometer observations are faulty, and are misleading in their results.

In view of all available data with which the writer is acquainted, he considers Mr. Cooper's specification right in theory and amply

* *Engineering News*, September 1, 1904, pp. 192-196.

† *Ibid. Engineering News.*

Mr. Goodrich. conservative in practice, provided a stipulation is added that equal suction effects are to be provided against on the lee side of sloping roofs, and that bridge floors, members, etc., are to be fastened so as to withstand the specified pressures from below as well as from all other directions. To be sure, the dead weights of most parts of structures are more than sufficient to overcome such forces in most cases; it is to be noted, however, that a sudden variation in atmospheric pressure of 1 in. of the barometric column (to which we are all accustomed when extended over a long period), will exert a stress of approximately 70 lb. per sq. ft. Some idea of the actual pressures resisted by high buildings can be easily secured by a few observations with any good aneroid barometer on the leeward and then on the windward side of such a structure. The sudden variation in pressure during the St. Paul storm would produce, under proper conditions, a pressure of 10½ lb. per sq. ft., aside from the effect of any impact due to wind velocity.

The theoretical investigation of the stresses due to a cyclonic vortex are most interesting, and explain many curious phenomena. The whole subject is one in which data are so scarce that all information, such as that contained in this paper, is of great value.

Mr. Turner. C. A. P. Turner, M. Am. Soc. C. E. (by letter).—In presenting a paper on the wreck of the Smith Avenue Bridge by wind, the writer's intent was to show by computation and photographs that the wind pressure involved, while within those limits usually figured on, acted in a manner not generally given due consideration in design, that is, in an upward direction. Indeed, it is this upward pressure, whether in the form of an upward current of air or the negative pressure on the exterior of buildings, that seems to cause the maximum destruction and damage to property. When a tornado passes directly over a dwelling-house, or similarly constructed building, it often happens that the walls are thrown outward with considerable violence, the wreckage appearing like the result of an explosion; proving that the atmospheric pressure outside of the building was suddenly reduced, and the building destroyed by the expansion of the air within.

Just after this storm, the writer visited some residences at St. Louis Park which had been reduced to a pile of wreckage no higher than his head, and was not a little surprised to learn that two families had escaped from such a wreck with but a few scratches.

Rough computation seemed to indicate that, if the storm could reduce the exterior barometric pressure from ¾ in. to 1 in., the internal pressure, of from 50 to 70 lb. per sq. ft., would, as a suddenly applied load, be sufficient to cause the destruction noted, since the framing and nailing are far better adapted to withstand inward than outward pressures. Unless pressures far exceed those noted,

Mr. Turner. it may be questioned whether such a storm would have a more disastrous effect on a modern steel skeleton or reinforced concrete structure than the destruction of windows and minor parts.

The wind pressure of the exterior air, at high velocity, under lower barometric pressure, combined with this explosive pressure, causes interesting conditions of stress. The writer noted a barn in which the leeward roof had been blown out, while the windward roof was left sticking up in the air, a case in which the explosive pressure had offset the external wind pressure on the windward side. A combination of these pressures working together may account for many curious freaks in the detailed wrecks of buildings.

Consideration of such pressures would point toward stiff construction in roof work.

In bridges, there is no confined volume of air, and one has to deal with the pressure on the truss members and on the floor. In the case of the Smith Avenue Bridge, the storm was cyclonic in character, the vortex traveling nearly normal to and above the bridge, and, judging by damage on the island below and in the city beyond, the writer would conclude that the storm center passed over the wrecked 250-ft. span between the center and the northwest end. Now, the probable breadth of the path subjected to severe pressure by the storm should be judged by its action at other points in its wake. At Minneapolis it picked up the flooring of the Tenth Avenue Bridge (wood joist and double planking, weighing from 25 to 30 lb. per sq. ft.), and dumped it into the river, for a length between 150 and 200 ft., leaving the remainder undisturbed.

Where severe damage was done by the storm, it seemed to be confined to narrow strips, though there were several of these in Minneapolis from 3 000 to 5 000 ft. apart.

These facts accord well with the views of Mr. Le Conte regarding wind pressure, while discrediting his tip-over theory.

Referring now to Figs. 1 and 2, Plate V, if the wind pressure had not been sufficiently great to lift the end of the 250-ft. wrecked span, how could the end of the span have been given a quarter twist while the truss was lying on its side?

The suggestion of Mr. Strobel, that the wind blowing against the bank may have been deflected upward, is untenable in view of lack of apparent damage to trees or foliage on the bluff.*

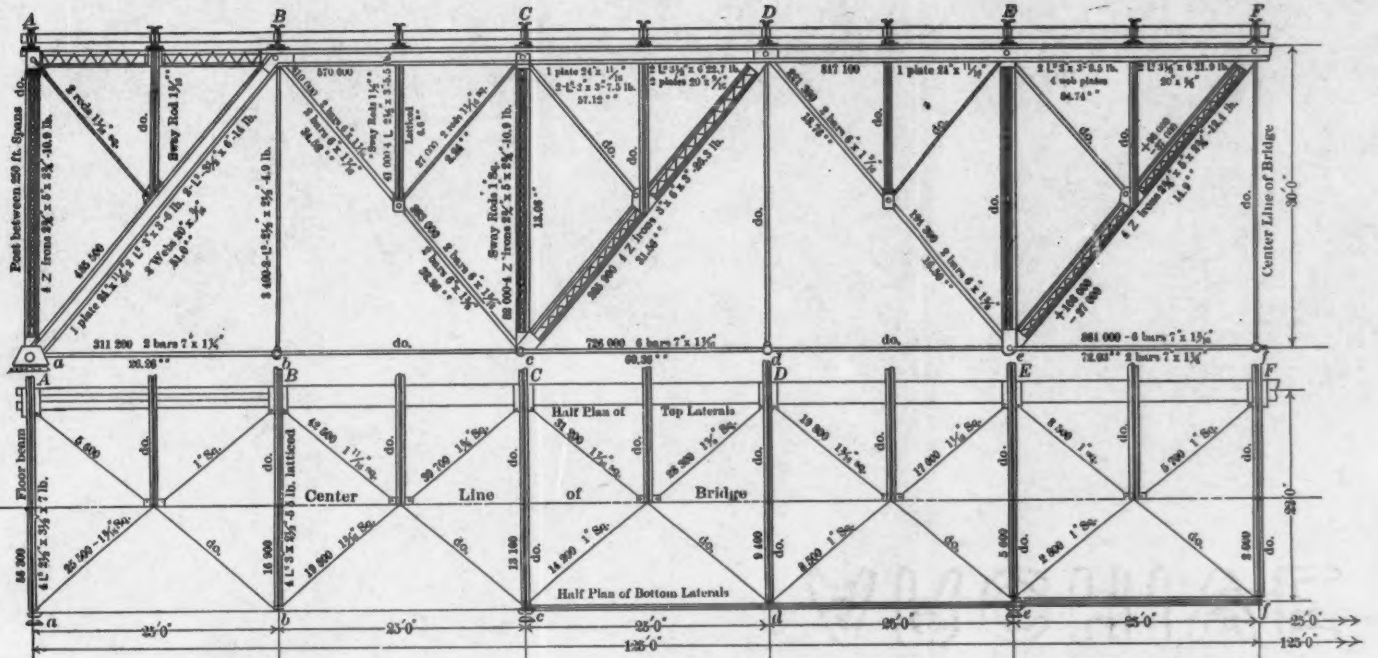
Were the wreck a plain case of tip over of the towers, the end of the 250-ft. span would unquestionably have fallen nearer the foot of the rocker bent.

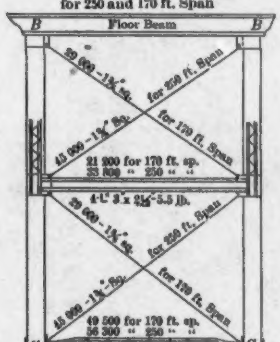
Referring to Plates VI and VII, it will be noted that the tower frame is twisted more than could reasonably be accounted for by a plain case of tipping over. Mr. Tanner† notes this, but tries to

* See letter of L. W. Rundlett. in *Engineering News*, Oct. 27th, 1904.

† *Engineering News*, Oct. 27th and Dec. 8th. 1904.

PLATE X.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 989.
TURNER ON
WIND PRESSURE ON BRIDGES.





Inclined End-Post Bracing,
for 250 and 170 ft. Span

STRAIN SHEET
FOR 250-FOOT SPAN
SMITH AVENUE BRIDGE
ST.PAUL, MINN.

Intermediate Sway Bracing
for 170 and 250 ft. Span

For 12' 6" Panel

Stringers {Web = $12\frac{1}{2}''$ x $\frac{5}{16}''$ Plate; Flanges 4-L^s 3" x 3" x 4 lb.}

Floor Beams {Web = 30" x $\frac{5}{16}''$ Plate; Flanges {4-L^s $3\frac{1}{2}''$ x 3" x 7.7 lb.; 4-L^s 3" x 3" x 6.6 lb. for overhanging bracket}}

account for it on the ground that the nuts or anchor bolts of the northwest pedestal were missing and the nuts were in place on the southwest pedestal. Bearing in mind the fact that the longitudinal bracing of the windward face of the tower is stronger than the two $1\frac{3}{4}$-in. bolts, this explanation seems to be untenable. Mr. Turner.

Referring now to Fig. 2, Plate VII, southeast of the southeast pedestal may be seen a large hole in the ground punched by the tower leg when it was twisted from its base and the bolts were broken off.

Were the failure caused by the tower tipping over, the leeward leg bases would be the axis of rotation, and the tower would certainly not have been jumped from its base as though twisted off by a long lever. In the paper the writer has referred to the bending and manner of breaking of the bolts, in support of his view of the failure, together with the condition of the roller rests on the rocker bent, and can see no rational theory of explaining the wreck other than that advanced.

As to the correct method of figuring, the writer agrees fully with Mr. Strobel that it is a very debatable matter, in view of present lack of experimental data on the subject.

In preparing the paper the writer was without exact data as to the sections of the truss, which were heavier than his estimate by 700 lb. per lin. ft., as shown on the strain sheet, Plate X. The area exposed was also greater.

Mr. Strobel is rigidly correct in his interpretation of Unwin's rule. However, if applied rigidly, it should cover not only the horizontal surface of the underside of the planking, but also the vertical area of the joist and plank ends and stiffeners, which would be 14 ft. $\times$ 30 $\times$ 125 $\times$ 33 $\div$ 22 = 78 700 lb., which has been left out of Mr. Strobel's computation.

A more exact estimate of the area of the truss members (for both trusses) is $9\frac{1}{2}$ ft. + $2\frac{1}{2}$ ft., for two hand rails, = 12 ft., and, for overturning, $\dfrac{12 \times 30 \times 125 \times 30}{22} = 61\,400$ lb. added direct uplift. Then 50 100 + 78 700 + 61 400, gives a total uplift of 190 200 lb. on the windward shoe.

Taking Mr. Strobel's statement of weight, we have 1 520 $\times$ 125 = 190 000 lb., an amount less than the uplift; in other words, if the wind blew as a gust, the wreck would result.

The problem of flight would be readily solved if a truly normal pressure on an aeroplane could be obtained, with no tendency for it to be carried with the wind causing such pressure, and the writer agrees with Mr. Strobel in his views as to resolution of actual wind pressures.

There is another item which is not covered in the foregoing com-

Mr. Turner. putation, and that is the suction on the lee side of the planking. Valuable information on this point can seemingly be obtained, as suggested by Mr. Goodrich, by making simultaneous observations of barometric readings on the windward and leeward sides of tall buildings during high winds.

It may be noted that the lateral force, considering the surface of stringers, is greater than that provided for by the specification; still, with the margin of safety in the working stress used, together with the sway system, it seems doubtful if the wreck would have occurred with a better detail of shoe on the rocker bent.

Regarding Mr. Strobel's reference to a competent engineer, in connection with the details of the structure, the writer is glad to say that he also has the pleasure of personal acquaintance with him, and entertains an equally high opinion of his work.

In the design of bridge and structural work, it is safe to say that no member of the profession has produced a structure that cannot be improved upon to a greater or lesser extent, and it is in the line of sound practice to benefit ourselves as we may by rigid analysis of observed failures of common specifications to meet the requirements of safety, economy and durability.

Replying to Mr. Gifford, a stiff riveted span, in place of the pin-connected span, would have enabled the single bolt in place on the shoe to have added approximately 50 000 lb. to the stability of the span against overturning, and, had the two bolts been in place, double this amount, instead of the insignificant amount noted; while the stiff chord would have enabled a large saving to be made in erection and would have been an economic investment, from that standpoint.

The writer is much pleased with Mr. Cooper's valuable discussion of wind pressure for long-span bridges, and agrees with his requirements except the first, which he would consider, in view of the importance of such a structure, should be taken as acting over a width of 150 ft. and, as seems to be Mr. Cooper's intent, that the excess pressure of 50 or 30 lb. for the respective lengths be combined with the 15-lb. pressure for the remainder of the span.

Regarding reported wind velocities,* the writer would call attention to evidences of eddy currents throughout the path of the storm, hence the readings of the anemometer should be accepted with caution, since, with the usual form of Robinson's cups, an eddy current of air having a rotary velocity of 60 miles might give a reading equivalent to a direct wind having a velocity three times as great.

* See Mr. Rundlett's letter to *Engineering News*.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 990.

THE RECLAMATION OF RIVER DELTAS AND SALT MARSHES.*

By J. Francis Le Baron, M. Am. Soc. C. E.

With Discussion by

Messrs. E. L. Corthell, L. J. Le Conte, Richard Lamb and J. Francis Le Baron.

In the following paper an attempt is made to show the practicability and great desirability of reclaiming the immensely rich swamp lands situated at the mouths of many rivers, notably the Mississippi, and the salt marsh lands lying along the whole seaboard, lands than which no richer exist on the continent, and which are pre-eminently adapted to most successful cultivation of, not only rice, but sugar and all classes of garden truck, these reclaimed swamp lands being of much greater agricultural value and capability than the irrigated prairies, which are so popular at present with rice growers in Louisiana and Texas.

The lands of the Mississippi Delta, being probably the largest single body of fresh and salt marshes in the United States, have been selected for examination and study, as being without doubt the most difficult of treatment, on account of the low range of the tides and the elevation of the river above the land to be reclaimed.

* Presented at the meeting of December 21st, 1904.

In carrying out the plan of explaining the treatment of the most difficult cases, from which it is easy to change the *modus operandi* to suit more advantageous conditions, the writer has based his computations on rice culture, as requiring more irrigation water, and conse-

MAP OF LOUISIANA
SHOWING LOCATION OF PRECIPITATION STATIONS
IN THE VICINITY OF NEW ORLEANS
ALSO OBSERVATION AND FORECAST DISPLAY STATIONS
IN OPERATION DECEMBER, 31st, 1903.
U.S. WEATHER BUREAU

FIG. 1.

quently more pumping, to irrigate or de-water. The plans can be readily modified to suit other conditions and localities.

The writer has lately been engaged to make examinations and to report on the reclamation of 500 000 acres of the Mississippi Delta lands.

LOCATION AND CHARACTER OF LANDS OF THE MISSISSIPPI DELTA.

The lands selected for discussion include the salt and fresh marshes and swamp lands in the Parishes of Plaquemines, St. Bernard, Jefferson, La Fourche, Terrebonne, Orleans, St. Charles, St. Mary, Iberia, St. John the Baptist, St. James, Assumption and Ascension. The conclusions and methods are also applicable to large portions of Vermilion and Cameron, and the lower parts of the Louisiana and Florida parishes, as well as all deltas and salt marshes. See Fig. 1.

Most of these lands are open fresh marsh, merging gradually into salt marsh at the southern end, and covered only with grass. They are entirely free from trees or bushes, except for a narrow margin along the bayous and a few scattering "chênières," or oak islands, and ridges in the marshes, which in Florida would be called "hammocks."

These lands are from 6 in. to 10 ft. above Mean Gulf Level, the great majority being about 2 ft.; and their drainage must be effected by pumping, as is the general custom in this region, where too low to drain by gravity.

The soil is composed of the rich alluvium brought down and deposited by the Mississippi River during past ages, and is inexhaustible in fertility. Probably no richer agricultural soil exists on the Continent of North America.

The writer caused numerous borings to be made, and tested the soil, personally, in several places. The borings, made for the New Orleans and Gulf Ship Canal and Locks, show it to consist of black clay, sand and silt for a depth of more than 80 ft., or as far down as the borings extended, intermixed in varying proportions, the silt being composed largely of vegetable matter. The State Commissioner of Agriculture says of these parishes: "The soil is exceedingly rich and productive."

METHODS OF RICE CULTIVATION.

In the cultivation of rice in the United States, two methods are now followed, the older being that in vogue in the lowlands of the Carolinas and Georgia. The modern method is radically different, and was first essayed in the new rice fields on the elevated prairies of western Louisiana and eastern Texas.

In the first case, large quantities of water are used in irrigating the

rice, and are considered absolutely necessary, the fields being low, swampy, and having to be embanked to keep them from being overflowed. In the later or modern method, comparatively small quantities of water are used. It is all pumped or obtained from artesian wells, and is used to irrigate the dry prairies. These fields are embanked to keep the water in. In the first case, the irrigation water is put on the fields by gravity, and, in most cases, has to be pumped off. In the latter, the water is generally pumped on, and is drawn off by gravity.

By the old plan, the watering extends over 95 days, whereas the modern practice proves that 68 days are sufficient, if the water is put on at the right time.

Methods of Reclamation.

The methods pursued in reclaiming land for rice, cane and vegetables are essentially the same. The important thing is to control the water supply and the drainage, protect the land from the overflow of salt water and crevasse water by ample protection levees, and make the lands long and narrow so that they can be worked by machinery and yet be well drained.

Lands close to the Mississippi River can be reclaimed naturally at less cost than those at a greater distance, as the irrigating water can be taken over the levee in siphons and put on the land with little trouble by short ditches. For lands farther away, wooden flumes must be built, in some cases, to cross intervening bayous, and, in some instances, pipes must be laid. As long as the supply is taken from the Mississippi River, either by siphons or by pumps, there will be no lack of water for these plantations, but the quantity to be pumped will be influenced largely by the rainfall, for, during some months, the rainfall on the tract may be sufficient without recourse to pumping at all, and, on the other hand, in those places so low that the water cannot drain off by gravity, it may be so much in excess of the needs of the crop as to make it necessary to pump it off. In other cases, the supply cannot be by gravity, but all irrigating water, as well as drainage water, must be pumped from the river, canal or bayou.

The daily rise of the lunar tide on this part of the Gulf Coast is 1.4 ft. When draining for rice, the water table in the ditches needs to be only 1 ft. below the ground surface, when drawn down; for

alfalfa, 2 ft.; for sugar cane, 3 ft.; and for garden vegetables and fruits, from 1 to 5 ft.

If the land is intended for rice, it will drain by gravity when the surface of the ground is 1½ ft. above low water, provided the drainage sluices are of the right size, and the rise and fall of the tide is not less than about 1½ ft. For alfalfa, the land must be 2½ ft. above low water; for cane, 3½ ft.; and for vegetables, from 2½ to 5½ ft., according to the variety. Lands below these levels will have to be kept dry by pumping during the cropping season.

Hygrometric Conditions.

Taking rice for example, the months during which water is required for irrigating, in the vicinity of New Orleans, are as shown in Table 1. This table also shows the quantities required, and the average length of time, with the mean rainfall, for the same months, being the mean of eleven circumjacent stations for the last six years (1897–1902).

TABLE 1.—Months in which, in the Vicinity of New Orleans, Water is Required for Rice Irrigation.

Month.	New Method.		Old Method.		Mean precipitation.	Remarks.
	Inches required.	Number of days to be supplied.	Inches required.	Number of days to be supplied.		
March	0	0	8	10	5.35	Water used in April is put on in March, in old method. For 4 out of the last 6 years there has been less than 2 in. rainfall in May, and for 2 years less than 1 in. in June. The means for New Orleans cover 32 years; for Houma, 12; for Lawrence, 10; Reserve, 1.
April	0	(30)	(4)	20	.....	
May	0	(31)	0	(30)	2.24	
June	4.77	4½	28	25	5.17	
July	17.98	13	45	31	6.83	
August	9.08	15	15	9	6.19	
Total, including rainfall.	31.83	32½	96	95	25.78	

By Table 1 it appears that, taking the average for the last six years, there has only been one month (May), during the rice growing season, when sufficient water has fallen for the use of the crop by the old

method, and but half of the time by the new method. Also, the new requires only 28% as much water as the old method, and requires only 34% as many days of pumping.

TABLE 2.—EVAPORATION OBSERVATIONS AT NEW ORLEANS, LA.

Authority: United States Signal Service.

Year.	Inches of Evaporation for:					Total for the year.
	April.	May.	June.	July.	August.	
1888	....	....	3 820	9.380	7.960	....
1888	....	....	3.790	....	....	....
1888	3.800	4.200	4.100	....	....	45.400*
1887	....	....	....	4.100	4.300	....
1889	....	....	8.200	8.700	8.900	....
Means	3.800	4.200	4.977	7.393	7.053	27.423†

* Computed for the fiscal year, 1887–88.

† 27.423 in. = total for 5 months' observation (153 days) = 0.179 in. per day.

Now, take the evaporation into consideration. Assuming that the means of the fragmentary observations of the U. S. Signal Service, which are all that are available, at this time, are approximately correct, and considering, also, that it is better to err on the safe side, it appears that the mean evaporation is greater than the mean precipitation in May, July and August, and about equal to it in June, while in April the precipitation is considerably in excess, as shown by Table 3.

The records for New Orleans alone, extending back for 32 years, show a mean annual rainfall of 57.54 in., and the mean for the last 8 years is only 49.63 in., while the mean for the first 8 years (1871 to 1878, inclusive) is 66.98 in. This seems to show cycles of about 20 years, the minimum having occurred in 1891, and the present time being on an ascending node.

Desmond FitzGerald and J. James R. Croes, Past-Presidents, Am. Soc. C. E., and Professor Russell agree that, in the latitude of New England and the Middle States, the evaporation from water surfaces is about equal to the rainfall, taking one year with another. Owing to the greater humidity in the South, it appears to be less. The evaporation for the entire year, at New Orleans, is given by the U. S. Signal Service at 45.40 in. in 1887–88. According to the observations of the

New Orleans Sewerage and Water Board, taken for a few months only, it would not exceed 2 in. The Signal Service observations agree more nearly with other authorities.

The quantity of evaporation given in Table 2 is 0.179 in. per day for the 5 months observed. The Signal Service officers, however, state that, in their opinion, their figures should be reduced 20%, making the quantity per day 0.143 in.

General Gillmore estimated, from observations taken on some open ponds in Florida, 0.300 to 0.250 in. per day.

The experiments of the United States Department of Agriculture, at Crowley, La., and in Texas, give a mean of 0.225 in. per day for 67.5 days.*

Evaporation is very largely dependent upon the wind, and this region is completely open to wind and sun. From experiments made by the U. S. Signal Service, it appears that, with the wind blowing with different velocities, the effects shown in Table 4 were produced, as compared with quiet air.

Hydrologic Conditions.

Owing to all these lands being below the level of the Mississippi River, with the drainage away from the river instead of toward it, and the lands themselves being a dead level, there is practically no watershed to be considered. Further, the level of the land is so low that there can be no loss of water by filtration, and the soil is so retentive that very little infiltration need be expected, even in those lands lying below the level of the water outside the protection levees. At least, that is the experience with reclaimed lands in the vicinity. Then, only the effects of rainfall and evaporation have to be considered.

The cultivation of rice requires more water than any other crop, therefore it has been selected for this discussion.

Rice cultivation, on the high, level prairie lands, like those of Crowley, La., and in Texas, and on the low lands, such as those of Georgia and the Carolinas, and those comprised in this belt, is very different as practised in the different localities.

In Crowley, La., and in Texas, the average length of the season during which water is used is only 68 days, during which time the water is turned on for only 10 days, while, in the low lands of Georgia

*Bulletin 113, Dept. of Agriculture, Office of Experiment Stations, pp. 23, *et seq.*

TABLE 3.—MEAN MONTHLY AND ANNUAL PRECIPITATION, IN INCHES, IN THE ZONE OF THE DELTA LANDS.

Station.	Period.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Number of Years Covered	Annual Precipitation.
Emilie	1897-1902	4.20	5.15	3.37	4.55	1.39	4.07	5.97	5.96	2.57	3.09	3.51	5.34	5	49.17
Southern University Farm	1897-1902	2.15	6.62	3.34	*5.75	2.38	*4.84	5.59	*4.11	*5.13	*2.86	*3.35	*5.60	3	51.72
Sugar Experiment Station	1897-1902	2.91	5.78	4.14	6.16	1.37	5.52	6.08	4.69	6.22	*2.72	3.40	5.71	5	54.65
Port Eads	1897-1902	3.07	5.53	3.71	5.69	1.47	*3.70	5.69	8.89	8.48	5.63	3.22	5.33	5	60.39
Schriever	1897-1902	3.27	5.65	3.56	5.48	2.69	7.08	7.98	6.57	5.17	3.55	3.27	5.47	6	59.74
Wallace	1897-1902	4.12	5.43	3.62	6.55	1.43	*5.96	6.34	6.90	5.37	2.47	3.96	6.52	5	58.67
Reserve	1901-1902	0.02	*5.13	2.72	6.44	2.52	2.37	7.62	3.58	3.68	2.25	3.89	*6.07	0	46.29
New Orleans	1870-1902	4.56	4.71	5.17	5.20	4.09	6.29	6.27	5.59	4.72	3.08	3.91	4.33	32	57.92
Houma	1890-1902	3.87	5.37	3.03	4.68	2.43	2.17	8.97	6.63	9.95	2.72	2.43	4.06	7½	51.31
Venice	1897-1902	3.24	5.37	3.97	4.08	1.94	6.03	7.62	8.41	8.21	4.41	2.93	4.52	5 to 6	60.73
Lawrence	1892-1902	2.68	6.30	3.88	4.33	2.90	6.88	7.09	6.76	5.44	2.79	2.77	4.03	8 to 10	55.85
Sums	....	34.09	61.04	40.51	58.91	24.61	54.91	75.15	68.09	59.94	35.57	36.64	56.98	....	606.44
	....	....	....	....	....	....	....	....	....	....	....	....	....	....	55.13
Means	....	3.09	5.55	3.68	5.36	2.24	4.99	6.83	6.19	5.45	3.24	3.33	5.18	....	4.51
	....	....	....	....	....	....	....	....	....	....	....	....	....	....	55.13

*Monthly record incomplete.

and the Carolinas, it is the custom to supply water at intervals during a season of 144 days, during which time the water is turned on for 95 days, the quantity in the former case being only 27.66 in., as against 96 in. in the latter, the rainfall being included in each case. The quantity of water furnished in the latter case would be only 29 in., if it were not for the fact that during the "harvest flow" the water is changed six times, or every 10 days, to prevent it from becoming stagnant.

TABLE 4.—EFFECT OF WIND ON EVAPORATION.

Velocity of wind, in miles per hour.	Evaporation, number of times greater.	Velocity of wind, in miles per hour.	Evaporation, number of times greater.
5	2.2	20	5.6
10	3.8	25	6.1
15	4.9	30	6.3

The "harvest flow," according to the practice in Georgia and the Carolinas, is kept on for 65 days, steadily, while in Crowley, La., and in Texas, it is only kept on for about 32 days.

These methods may be designated the new and the old practice. The Crowley and Texas method is the new, and the other the old. The former marks a new era in rice growing, and has exploded the old and erroneous ideas that enormous volumes of water are needed to irrigate rice and that evaporation is excessive along the Gulf Coast.

As a matter of fact, numerous observations prove that it is less there than in many other places.

The mean rainfall for July and August is 1.59 in. more on this tract than at Crowley, while the total annual rainfall on this tract is 2.34 in. more, taking the mean of 14 years.

It has been demonstrated conclusively that abundant crops of rice can be raised on the new plan, and, therefore, the writer has adopted it in his estimates of water and pumping.

In the new method no water is put on, from the time of planting, for 2 or 3 months, or up to about the middle of June, dependence being placed on the rains to sprout the seed.

It appears from Table 1 that water is supplied to the rice on 32½ days, but part of this is rainfall; so that, leaving out the rainy days, there are only 10 days in the season when pumping is required,

in years of normal rainfall. This includes also the quantity lost by evaporation. Therefore, it appears that the pumping of water for irrigation, even if all the water had to be pumped, would not be a matter of very heavy expense, even for rice, and it is less for all the other crops.

Next, consider the length of time during the rice-growing period when siphons can be used. This will depend largely on the location, for, the lower this location is down the river, the less the river is elevated above sea level and the level of the marshes, and, therefore, the less the fall for the siphons and the shorter the time during which they can run.

Table 5 shows the highest and lowest water and the mean elevation, above Mean Gulf Level, of the Mississippi River at New Orleans for 8 years. Also, the number of days in each month that the river was 1 ft. or less above Mean Gulf Level. This table is taken from the hydrographs of the river, made in the office of the State Engineer of Louisiana, from the Bulletins of the U. S. Weather Bureau.

TABLE 5.—HEIGHTS OF THE MISSISSIPPI RIVER, AT NEW ORLEANS, ABOVE MEAN GULF LEVEL, FOR THE YEARS 1890 AND 1897 TO 1903, INCLUSIVE.

Last day of month.	Lowest.	Highest.	Mean.	No of days 1 ft. and less.
Oct. 31	0.8	4.9	2.2	1¼
Sept. 30	1.0	5.4	2.7	0
Aug. 31	1.7*	9.7	3.9	0
July 31	2.5	13.7	5.5	0
June 30	3.9	16.3	8.4	0
May 31	3.9	17.8	11.7	0
Apr. 30	8.2	18.6	13.9	0
Mar. 31	2.00†	18.6	10.6	0
Nov. 30	0.7	4.2	2.2	10
Dec. 31	0.6	11.4	3.0	7½
Feb. 28	2.1	14.8	7.9	0
Jan. 31	1.3	12.8	5.5	0
Mean	2.4	12.3	6.4	....

* 1900. † 1897.

Table 5 shows that there is not a day during the rice-growing season, from March to September, that the river is not 1 ft. or more above Mean Gulf Level in the vicinity of New Orleans, the lowest

water ranging from 1.7 ft. in August to 2.0 ft. in March, and the mean height averaging from 3.9 to 13.9 ft. above Mean Gulf Level.

Theoretically, a siphon should run if the water in the river is only a film higher than that in the marsh, but, in practice, it is found that it is impossible to make joints so tight that some air will not leak in. Also, air is disengaged from the water, and it is stated by prominent engineers who have had experience with siphons in that locality, where they are in common use to take water from the river, as far down as the writer went, or about to Buras, than 1 ft. difference in level is about the practical working limit.

Table 6, taken from the records of the U. S. Mississippi River Commission, shows the number of days, with dates, when the river has been 1 ft. or less above Mean Gulf Level at the Carrolton Gauge, New Orleans, from 1850 to 1902, inclusive.

TABLE 6.—Number of Days when the Mississippi River, at the Carrolton Gauge, Has Been 1 Ft. or Less above Mean Gulf Level.

Date.	Lowest.	Date.	Lowest.
Nov. 18, 1850	−0.63	Oct. 24, 1885	+0.77
Nov. 24, 1851	+0.07	Nov. 27, 1886	−0.53
Feb. 4, 1852	+0.47	Nov. 20, 1887	−0.83
Dec. 30, 1854	−0.13	Jan. 10, 1888	−0.33
Nov. 5, 7, 9, 1858	−0.53	Oct. 16–29, 1888	−0.03
Nov. 13, 1859	−0.58	Nov. 9–11, 1889	+0.27
Oct. 18, 1860	−0.83	Oct. 21, 1891	−0.18
Dec. 27, 1872	−1.73	Nov. 21, 1892	+0.02
Nov. 30, 1873	−0.21	Dec. 8, 1893	+0.17
Nov. 24, 1874	−0.23	Nov. 15, 1894	−0.18
Nov. 13, 1875	−0.23	Jan. 1, 1895	−0.13
Dec. 30, 1876	−1.33	Nov. 9, 10, 12, 24, 1895	−0.08
Jan. 9, 1877	−1.53	Oct. 9, 11, 1896	+0.62
Oct. 7, 22, 24, 1877	−0.13	Nov. 27, 1897	−0.03
Nov. 22, 1878	−0.23	Jan. 1, 2, 1898	+0.87
Nov. 24, 1879	−0.93	Nov. 4, 1899	−0.13
Nov. 2, 1880	+0.17	Jan. 21, 1900	+0.67
Sept. 10, 1881	+0.17	Dec. 18, 1901 Dec. 21, 1901	−0.14
Nov. 8, 1882	+0.87	Mar. 17, 1902	+0.97
Oct. 15, 1883	+0.37		
Dec. 2, 5, 1884	+0.17		

Table 6 shows that for the last 52 years there have only been 53 days when the river was as low as 1 ft. or less above Mean Gulf Level, or about 1 day per year. It also shows that these low stages have occurred in the months of October, November, December and January, with two exceptions, one of which was in September and the other in February, all being months when no water is required for rice, unless

it should be attempted to raise two crops, as is sometimes done. In that case the river records show that in 52 years a total of 30 days might be expected when the water would be too low to siphon in November, 10 days in December and 6 days in January, or, on the average, a little more than ½ day in November every year with a possibility of 10 days, 1 day in December every 5 years with a possibility of 7½ days, and one day in January in every 8½ years.

The study of these records shows conclusively that siphons can be depended upon for irrigation in the vicinity of, or above, New Orleans, the year round, and no pumps will be required. This supposes, of course, that the ditch or irrigating canal is brought up to the levee on the land side, with the water at Gulf Level. The mean rise and fall of the tide at Grand Pass is 1.4 ft. If the canal is fairly straight and unobstructed the rise of the tide at the levee should be about 1 ft., which would give a 5-in. fall at low water, and would give sufficient grade for the canal to discharge on the land at all times from half-tide ebb to half-tide flood, a period of nearly 6 hours, on the average, but variable. Land which is 1 ft. or more above Mean Gulf Level could not be covered at the extreme low-water stage of the river. The grade would be greater at spring tides, as the tide falls lower.

The level of the Gulf is influenced greatly by the winds, and sometimes it falls nearly 2 ft. below the Mean Low-Water plane, owing to a long succession of northerly winds, which lower the water near the shore. On the other hand, it has once risen 6.3 ft. above Mean Gulf Level, or 7.0 ft. above the plane of Mean Low Water, owing to an extraordinary storm. Of course, at such a time the siphons would not work, but, as such high waters always occur during storms and are accompanied by rain, no irrigation water is needed. When the storm subsides the water recedes and the siphons resume their work.

Table 7 shows the monthly means of the highest and lowest water at Ft. Jackson, and the number of days it was down to only 1 ft. or less above Mean Gulf Level, during the rice-growing season of 6 months, March to August, inclusive, for the years 1897 to 1902, inclusive, collated from the Mississippi River Commission's daily gauge readings.

Table 8 shows the extreme high and low water above Mean Gulf Level, at Ft. Jackson, for the whole year, from 1891 to 1902, inclusive, from the reports of the Mississippi River Commission.

TABLE 7.—MONTHLY MEANS OF HIGHEST AND LOWEST WATER IN THE MISSISSIPPI RIVER AT FT. JACKSON, DURING SIX MONTHS OF EACH YEAR FROM 1897 TO 1902, INCLUSIVE.

Year.	March.		April.		May.		June.		July.		August.	
	H.	L.	H.	L.	H.	L.	H.	L.	H.	L.	H.	L.
1897..........	5.1	2.5	5.5	5.0	5.5	4.9	4.9	1.3	1.9	0.5	1.5	0.1
1898..........	3.8	1.7	5.0	3.6	4.9	4.0	4.6	2.3	3.0	0.4	2.4	0.3
1899..........	4.8	2.5	5.3	4.5	5.1	3.8	4.1	2.7	3.0	0.9	1.7	0.3
1900..........	4.1	2.1	4.2	3.1	4.0	2.0	3.1	1.2	3.9	0.9	2.3	0.1
1901..........	3.1	0.3	4.1	2.7	4.3	2.2	3.5	1.4	2.5	0.5	5.5	0.0
1902..........	4.5	0.4	4.7	4.0	4.1	1.1	2.4	0.9	2.3	0.9	2.3	0.7
Means........	4.23	1.58	4.80	3.82	4.65	3.00	3.77	1.63	2.77	0.68	2.61	0.25

Number of days the water was as low as 1 ft. or less above Mean Gulf Level; and mean of 5 years.

Possible number of days in 1 year... ...	14	0	0	0	1	20
Mean number per year.....	3.6	0	0	0	0	10

TABLE 8.—EXTREME HIGH AND LOW WATER IN THE MISSISSIPPI RIVER AT FT. JACKSON, FOR 12 YEARS, FROM 1891 TO 1902, INCLUSIVE.

Year.	Date.	Above Mean Gulf Level, in feet.	Date.	Below Mean Gulf Level, in feet.
1891..............................	Mar. 31	5.22	Nov. 30	—1.18
1892..............................	June 13	5.17	Nov. 20	—1.18
1893..............................	June 15	5.02	Dec. 7	—1.38
1894..............................	Apr. 8–9 Oct. 8	4.12 6.32	Nov. 12–14	—1.38
1895..............................	Apr. 2–3	2.92	Jan. 9 and Dec. 18	—1.28
1896..............................	Apr. 19	4.22	Dec. 4–5	—0.78
1897..............................	Apr. 22, 23 Apr. 29 May 14–16	5.52	Dec. 8	—0.83
1898..............................	Apr. 23, 26	5.02	Jan. 2 Dec. 26	—0.83 —0.78
1899..............................	Apr. 21	5.30	Dec. 4	—0.80
1900..............................	Apr. 20–28	4.22	Jan. 14	—0.43
1901..............................	May 8, 9, 18	4.32	Nov. 25	—0.98
1902..............................	Apr. 14	4.67	Feb. 3	—0.28
Sum..............................	23 times	109.04	16 times	15.10
Means..............................	....	4.31	....	—0.94

Table 8 shows the average height of extreme high water at Ft. Jackson to be 4.31 ft. above, and the average height of extreme low water to be 0.94 ft. below Mean Gulf Level. These are the means of the extremes for a term of 12 years, covering the whole of each year.

In explanation of Table 7 it must be understood that at this station two readings of the gauge were taken daily, and this accounts for the apparent discrepancy in some parts of the table, as, for instance, in the July column of low water, which shows a mean for 6 years of only 0.68 ft. above Mean Gulf Level, and yet the lower part of the table shows that in July there were no whole days as low as 1 ft. or less above the datum, with a single exception. The explanation of this is that during a part of the day the water was less than 1 ft. high, but at some time during the 24 hours it was more than 1 ft. high. This is owing to the backing up or raising of the river water by the daily action of the tides. The salt water, being heavier, runs up on the bottom, while the fresh water flows over the top. This tidal influence is felt as far up as Red River Landing, about 225 miles above New Orleans.

The water at New Orleans has been slightly brackish once in a period of about 10 years. At Harvey's, at the head of his canal, the tide rises and falls about 2 in. daily, and in storms about 1 ft., but the water is perfectly fresh. At New Orleans the rise and fall in the river is about the same.

The great Gulf storm of August 14th, 1901, producing the highest rise ever known, raised the river 4.9 ft. at Ft. Jackson, 5.2 ft. at New Orleans, and 1.00 ft. at Red River Landing, above the river height at the time. When the proposed New Orleans and Gulf Ship Canal is built to its full width and depth, the daily tide will probably be about 10 or 12 in. at Harvey's.

Table 7 shows that at Ft. Jackson the greatest height of the water, in the six growing months, for the last 6 years, was 5.5 ft. above Mean Gulf Level, and the greatest mean for any one month was 4.80 ft., for April. The lowest water was 0.3 ft. below Mean Gulf Level in March, and the lowest mean height for 6 years was 0.25 ft. for August. The mean of 6 years' observations shows that in March there are 3.6 days when the water is 1 ft. or less above Mean Gulf Level, and in some years there are 14 days; that in August there are 10 days, with possibly 20; and, once in 6 years, there is 1 day in July.

These conditions, as far down as Ft. Jackson, make the raising of rice by the use of irrigation water obtained from the Mississippi River by siphons, not perfectly satisfactory. The mean of 3.6 days in March, with a possible 14, is not material, as no irrigation water is wanted in March, April or May, provided there is any rain at all, but a loss of water for a mean of 10 days in August, with a possible 20 days, might prove disastrous if it came in the first part of the month, when the water is needed for the "harvest flow."

An examination of the gauge readings for the past 6 years shows that for about three-fourths of that time the scarcity occurred in the last half of the month; the other times occurred intermittently. Therefore, there would not be much danger for the crop, and the writer has observed that rice is grown successfully by the use of siphoned water as far down as Socola. It would be impossible, however, to raise two crops per year as far down as Ft. Jackson, by using siphon water.

Method of Drainage and Irrigation.

The land should be first surveyed and divided into sections of 1 mile square, and these subdivided into 160, 80, 40, and 20-acre lots, to suit purchasers. A protection levee, from 8 to 9 ft. in height above low water, should be built around each section. This is rendered necessary because on one occasion the water of the Gulf, during a severe storm, was raised by the wind 7.0 ft. above low water; and this may occur again. This is said to have been the highest water known since the settlement of the country, and, therefore, the levees should be 8 ft. high in the protected places and 9 ft. high in locations more exposed to the waves. Where the surface of the marsh is 2 ft. above low water, this will make the height of the levee above the ground surface from 6 to 7 ft. The small levees, inside the protection levee, need only be high enough to flood different lots about 1 ft. deep, making the height of these sub-levees from about 1½ to 2 ft. above the level of the ground.

Each mile section should have a marginal canal or "face ditch" around it on each of its interior sides. This ditch should be 3 ft. wide at the bottom, and about 3 ft. deep, with sloping sides. The 80-acre and smaller lots should be bounded by ditches, 9 in. at the bottom, 3 ft. at the top, and 3 ft. deep.

When the shape of the land will admit, these sections should be

laid out in groups, 4 miles wide by 8 miles long, and a protection levee built around each, with a canal for drainage, and another canal longitudinally through the center of the tract. This arrangement will reduce the expense greatly, by dispensing with the high protection levees around each section, as would be necessary if treating one section singly. The arrangement is shown by Fig. 2.

Wooden sluice boxes, with gates, can be laid through these levees, to admit the irrigation water at high tide and permit the drainage water to escape at low tide. This arrangement is feasible in the fresh-water marshes, where the tide rises and falls daily, as the water is not salt. In most places, however, in these lands, the irrigation water will have to be brought in a special canal or ditch from the Mississippi River, or, where that is impracticable, pumped from the bayou.

Each case requires to be made the subject of separate study and treatment, but is perfectly feasible, and each is now being worked at a profit in Louisiana and in the Carolinas and Georgia. The most unfavorable case would be where the irrigation water had to be brought a long distance, in pipes or flumes, and then pumped out after irrigating the land. This might occur in some of the salt-marsh lands, a long distance from the river.

Another case requiring more pumping would occur in the lowest of the fresh-water marshes, where the same pumps would be used to pump in the fresh water for irrigation, and afterward pump it out.

The cost of this pumping, however, would not be as great as might at first be supposed. In the first place, the pumping operations for cane and rice would only extend over from 50 to 70 days, unless it was desired to raise two crops of rice, and, during that time it would not be continuous, by any means. In the case of garden vegetables, it would depend on what was being raised, but, in any event, would not extend beyond the crop-growing period, whatever that was.

In the second place, the lift would be very slight, often not more than 1½ ft., which would permit the use of the cheapest class of pumps, and even wheels alone in many places. Thirdly, the location, for the most part, is a perfectly open and wide expanse of flat marsh, without trees or bushes, except an occasional clump of low trees

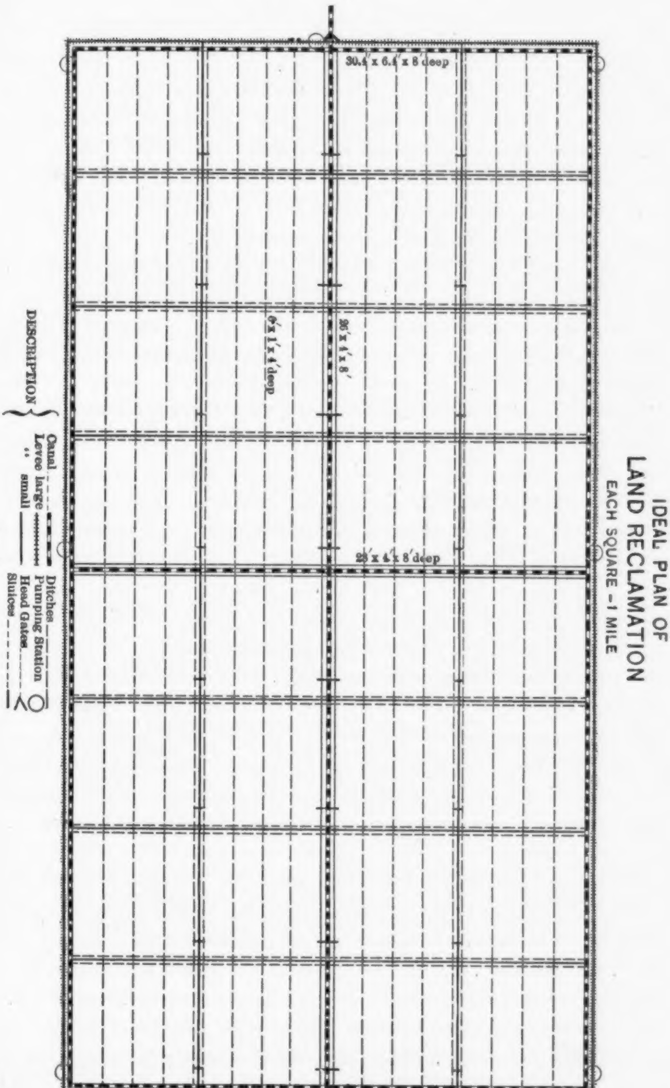

IDEAL PLAN OF
LAND RECLAMATION
EACH SQUARE – 1 MILE
30.4' x 6.4' x 8' deep
6' x 1' x 4' deep
26' x 4' x 8'
25' x 4' x 8' deep
DESCRIPTION
Canal
Levee large
" small
Ditches
Pumping Station
Head Gates
Sluices

FIG. 2.

covering possibly an acre, and a few scattered trees on the borders of the bayous. Therefore, as it is fully exposed to the sweep of the wind from all directions, as much as if at sea, windmills can be used advantageously, as they are on the Zuyder Zee in Holland; indeed, several may be seen on the line of the New Orleans, Ft. Jackson and Grand Isle Railroad, which runs through this territory. The cost of pumping by windmills is very low, consisting chiefly in repairs to the mill and pump, and the daily oversight by an oiler, who could be the watchman tending the sluices, embankments, etc.

The cost of oil and repairs would not be more than 10% of the cost of the mill, which may be taken at $300 for the larger sizes, set up.

An objection to windmills, besides the uncertain character of their power, is the small size and small duty of the commercial mills found in the United States.

One of these mills, 16 ft. in diameter, with the wind at a velocity of 20 miles per hour, and with a lift of 30 ft., will pump 4 224 gal. per hr., which is only about one-fifth of 1% of the capacity of a 42 by 16-in. Menge pump, and one of the latter will be required to irrigate every 320 acres, provided a case should occur where all the water has to be pumped to the fields. It would require about 473 of these windmill pumps to do the same work, or one pump to 0.68 acre, that is, provided all the water of irrigation has to be pumped, which would hardly ever be the case; but it serves to show the comparative duty of the windmill and electric or steam pump.

The experiments made under the auspices of the United States Geological Survey* show tests made of nearly all the windmills on the market in the United States. According to these tests, the 16-ft. "aermotor" appears to be among the best, and this never exceeds 2 h.p., with a 20-mile wind, and, with a 15-mile wind, may be assumed to be only 0.8 h.p.

The old-fashioned Dutch windmills, with a diameter of 70-ft., yield 7 h.p., with the same wind, according to the experiments of Coulomb in Holland. This would equal one mill for about every 5.9 acres, for irrigating, on the same basis as computed previously, and the ratio of the Menge pump to these windmills would be about 1 to 54.

The cost of one of these large Dutch mills would not fall much short of $800, therefore, windmills would be vastly more expensive

* "Water Supply and Irrigation Papers, Nos. 41 and 42."

than electric pumping, and there would be some uncertainty as to their action. However, for use in small separate fields, or as an auxiliary, they would be economical. In cases where the irrigation water is supplied by gravity, and the pumps are only required to take off the surplus water, one 16-ft. aermotor will de-water 1.38 acres, and one Dutch mill about 11.8 acres per day.

Mr. J. B. Watkins, reporting on the methods pursued in reclaiming large areas of tide marshes in Louisiana, says:*

"Our plan of reclamation is to build dikes, along the Gulf, rivers, lakes and bayous, of sufficient height and strength to prevent overflow of each in the event of floods from rain and storm tides, and in this we will be materially assisted by the natural levees found in many places along these waters.

"We cut, parallel to each other, and ½ mile apart, canals 18 ft. wide and 6 ft. deep. At right angles with these, at intervals of 2½ miles, we cut larger canals, thus forming the land into oblong blocks, ½ mile by 2½ miles, each containing 800 acres. Across these blocks, at proper intervals, we cut lateral ditches, 30 in. deep by 8 in. wide at the bottom, flared to 30 in. wide at the top.

"These canals are cut, the levees formed, and the dikes are, to a considerable extent, built by the use of powerful floating steam dredges. The smaller ditches are cut by ditchers propelled by steam power, passing through but once, at the rate of 1½ miles per hour. At the proper localities, we erect automatic flood gates, by means of which we control the stage of water in the canals, and the necessary volume of water is regulated to some extent by the ebb and flow of the tide. This is supplemented by the use of powerful wind pumps, and when the natural elements will not accomplish the work, we readily move upon the canals to the spot our ditching, plowing and cultivating engines and attach them to pumps. Thus arranged, with control of the water, these blocks of land are in condition for the most successful rice culture.

"Rice may be planted any time from February to June, very much the same as wheat, and upon ground similarly prepared. When it has reached a growth 2 in. high, water is let in upon it and the ground gradually flooded; care being taken not to cover any of the plants with the water. The land is kept flooded sufficiently to kill all the grass and weeds, until the rice is about 18 in. high. It has then sufficient start to choke down any foreign growth, and the water may be drawn off and the ground allowed to become dry and firm for harvest time, which may extend over several months, according to the

* "Tide Marshes of the United States," Special Report No. 7, U. S. Department of Agriculture.

time the seed is sown. Rice is harvested and threshed with the same kind of machinery as used for wheat."

His dredges have a capacity of a mile of canal, 6 by 18 ft. per month each, and he plowed 70 acres of land per day with gang plows drawn by traction engines.

In proportioning the canals for supplying irrigation water, the quantity of water required is determined by previous experience in rice growing. This must be increased by the quantity lost by evaporation, both in the field and supply canal, and this, in turn, must be diminished by the average recorded rainfall in that vicinity. In a tract as large as this, extending fully 100 miles north and south, and east and west, the hygrometric conditions will vary, and the computations adaptable to a plat at the west might differ considerably from those for one at the east end. Again, if the supply canal is long, more water will be lost by evaporation than if it is short. Therefore, each case must be made the subject of special study.

From experiments conducted by the U. S. Department of Agriculture, at Crowley, La., it appears that the depth of water required on the rice field during a season of 64 days, during which time the water was turned on for 12 days, was 26.51 in., of which 10.04 in. were rainfall; and 14.47 in. of this were taken up by evaporation, leaving the net depth of water received by the land 12.04 in.

At Raywood, Texas, similar experiments by the same parties gave 28.81 in. of water required by the land, of which 19.66 in. were supplied by the pumps, and 9.15 in. were rainfall. Of this, 16.03 in. were taken up by evaporation, and the net depth of water received by the land was 12.78 in., extending over a period of 71 days, during which the water was turned on for 18 days. These two experiments show a very close coincidence, and furnish valuable data for computing the water required in similar locations.

Where the water has to be brought a long distance, from the river or Harvey's Canal, reservoirs can be easily made, in many cases, and these will greatly reduce the length of the pipe or flume, and often cause it to be dispensed with entirely. In making these reservoirs, advantage would be taken of one of the many small bayous or creeks, of which there are a great many running through the marshes in all directions. These would be dammed at the point nearest the land to be watered, and the salt water, if any, pumped out. A ditch or flume would then be built at the upper end to bring in the fresh water, and

the creek or bayou would form the reservoir. It would be necessary to run protection levees along both banks to keep out storm water.

Accurate topographic and hydrographic charts of these marshes are necessary in order to locate and plan these reservoirs and the system of levees and canals.

Methods and Cost of Pumping.

The cost of running the pumps will vary chiefly according to the fuel used, as colored men, working for very low wages, can be found who are capable of attending to them, as is done now. The U. S. Department of Agriculture has made tests of, and has reported on, the cost of fuel per acre irrigated in Louisiana. The following is quoted from this report:

"Three kinds of fuel are used to make steam for irrigation pumps in the rice district: coal, wood and oil. Coal is the most expensive because of the long hauls necessary, and oil, based upon the experience of the one year that has passed since the Beaumont oil basin was discovered, is by far the cheapest and most satisfactory. Pittsburg, Kans., bituminous coal sold as high as \$4.75 per ton,* and wood at \$1.50 and \$3.00 per cord, while the oil delivered f. o. b. in car lots, cost from 48 to 62½ cents per barrel. Based upon reports received, the cost of fuel, necessary to irrigate an acre of rice, was between 60 cents and \$1.00 when oil was used, between \$2 and \$3 per acre when wood was used, and fully as much for coal as for wood. Crude mineral oil has proven a most satisfactory fuel. A uniform and high pressure in the boilers is easily maintained, and one fireman can easily handle a battery of half a dozen or more large boilers. The combustion is practically complete, and no injury to the boilers from the hot blast has yet been noted."

Fuel oil is now 75 cents per bbl. (42 gal.) in tanks on cars at New Orleans, and 12½ cents per bbl. for less than 10 000 gal. per month, delivered in the consumers' tank, by pumps, or 9 cents per bbl. for quantities of more than 20 000 gal. per month, the latter being brought in tank ships from Texas.

Two styles of pumps are commonly used about Crowley for irrigating, the rotary and the centrifugal. The former gives more efficiency, but it is heavy and requires very solid foundations as it is geared directly to the engine shaft, and therefore no settlement is allowable. The centrifugal, on the contrary, is generally run with a belt, and considerable settlement will not derange it. It is light, and

*This was at Crowley, La., in 1901. It is now (1903) \$4.25 a ton, delivered on lighters in New Orleans; Alabama coal, \$3.

easy to keep in repair. A variety of Archimedean screw, made in New Orleans, called the Menge pump, after the inventor and maker, is probably the best for the low lifts that prevail on this land.

Rainfall and Infiltration.

In computations based on rainfall, it is of the first importance to have as many stations as possible in the tract or around it, as the precipitation varies greatly even in as comparatively small an area as that of the City of New Orleans. In this case, fortunately, there are numerous stations almost surrounding the tract, so that a very fair general average of the rainfall can be obtained. The monthly precipitation at each station for a period of 7 years shows that the mean precipitation on this tract varies from 2.24 in. in May to 6.83 in. in July.

On part of this tract the water that is put on for irrigation must be pumped off, minus what has been dissipated in evaporation and absorption. All this soil is so retentive and the grade is so low that there will be no loss by filtration, or seepage. The gain by infiltration will be very slight, judging by the experience of the planters and those living, or cultivating, immediately back of the river levees. This is owing to the very retentive, puddle-like character of the soil.

The late John M. Goodwin, M. Am. Soc. C. E., one of the commissioners of the Pennsylvania Ship Canal, estimated the infiltration and leakage in that canal as 20% of the entire volume. Of this, the leakage would probably constitute about 75%, leaving only 25% for infiltration proper, or 5% of the entire volume of water. The experiments reported by General Gillmore, of filtration in Florida canals, give a very much greater quantity than would be expected on this land, owing to the sandy and extremely porous character of the Florida soil. In the absence of any known experiments in this line in this neighborhood, the quantity can only be approximated, but it is safe to assume that it will not be large.

An analysis of the rainfall from 1872 to date and Table 3 show that the mean annual precipitation is 55.13 in.; that there is no well-defined rainy or dry season; that the mean monthly precipitation is 4.51 in.; that the greatest monthly precipitation, in any one year during the last 6 years, at any one of the surrounding stations, was 19.55 in.; that the mean of the greatest and least monthly precipitation for each year for 6 years is 10.77 and 0.79 in., respectively.

It also appears, by the table of observations taken by the New Orleans Drainage Board, that from 1881 to 1894, inclusive, there occurred one rainstorm of 3.60 in. in 1 hour, and several others of shorter duration, but nearly equal intensity.

From the "Table of Excessive Precipitation" furnished by the Weather Bureau, and extending from 1870 to date, it appears that there has been as great a fall as 9.22 in. in 2 days, and 7.40 in. in 1 day.

Table 9 shows the months and days of the month when excessive precipitation has occurred during the last 32 years, arranged to show the months of greatest excessive precipitation. This table has been collated from the records of the U. S. Weather Bureau and the observations of the New Orleans Drainage Board, at New Orleans, Lawrence, Houma and Venice.

From Table 9 it appears that the months when short falls of great precipitation occur most often are, in the order named: April, September, February, March, August, June, July, December, October, May, November and January. Of the six most pronounced months, four are months of rice growing. Therefore, there may be an occasional engorgement of the drainage system during these months, for a short period. The flooding of rice for three or four days, or even several weeks, at some periods of growth, will not prove harmful.

The most excessive precipitation of this kind occurs in April, twenty-four storms, each with a mean precipitation of 4.02 in., having occurred in the last 32 years. These storms may last one or two, and, in rare instances, three days. This analysis shows that there is not much to fear from these short storms of excessive rainfall. The four months of most excessive precipitation are months when no water is put on the rice, dependence being had entirely on the rainfall, which is generally sufficient. If there should be a drought, the water would have to be turned on; therefore, when these rainfalls occur in March, April or May, they will be an advantage. Those that occur in June, July or August would cause an engorgement if they happened to come just after the water had been let on, and might necessitate starting up the pumps, to relieve the land of the surplus. On the other hand, if they occurred just before the water was due to be let on, they would save the trouble of opening the gates or the cost of pumping so much water, in fields where pumping was required for

TABLE 9.—EXCESSIVE PRECIPITATION, IN INCHES, AT POINTS NEAR NEW ORLEANS, LA., DURING THE LAST 32 YEARS.

Day.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1			3.02									
2							3.49		4.66 3.25 3.78	2.75 2.95	2.61	
3	2.82			6.01 3.80		2.90	2.48 3.25		1.24 3.40		3.28	
4						2.70	7.52				2.61	3.82
5	2.62			2.96	2.00 3.98		3.38					
6						3.02 2.00 2.86	1.40				2.99	
7				5.51	4.06		1.00 2.66	1.62			4.39	
8	4.07		3.36	9.22								
9			2.78			2.68		4.30				
10	2.72 3.12 2.85					3.50 3.68			3.01 5.27	3.47	2.70	
11		2.60 2.72	2.75					3.10 2.77	3.08 2.77	4.15		
12		3.14					3.00		9.80		2.80	1.26 2.85 3.40
13							1.15	2.56 2.60	3.97			
14		3.68	3.95 4.04 3.11	3.25				4.14	1.20		2.32	
15	4.28	2.60 4.02	8.66				1.25	3.70 2.67 3.85	2.89			2.92
16		5.10		2.82 3.84	3.39		3.11			2.63		
17		3.00 5.76 2.78		1.70 4.00 3.93					2.52 7.22			
18				5.92 2.68		2.70	2.01 4.07	4.25				2.54
19	3.71	5.25 5.71	3.98	2.88	3.54	1.04				3.19		
20		2.60 3.59 2.72		2.90	3.29			8.90	2.78			
21							2.74	4.08	3.50	2.55		
22				7.49		1.25				2.00		
23			3.60	4.84 3.59		3.08		1.25		4.13		
24		4.21	2.69 2.60 4.50	2.18 2.05 7.40	4.56				3.01			
25				1.35			2.61	2.80	1.39 3.60 5.28 3.23	2.54		
26	2.71					2.86 4.44					3.35	
27			3.72		4.49			2.84				

TABLE 9—(*Continued*).

Day	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept	Oct.	Nov.	Dec.
28	......	......	......	2.72	......	......	......	......	2.88 2.85	2.66	......	3.27
29	......	......	3.61	......	......	3.20	......	......	......	3.52	2.65 2.52	2.90 3.40
30	......	......	......	3.53	5.10	5.46 2.76	......	......	5.60	3.04	......	......
31	......	......	......	......	......	......	......	......	......	......	......	11.35 2.70 2.59
Sum	28.00	59.43	56.52	96.57	34.44	50.15	45.12	55.45	91.57	39.58	32.17	43.00
Number of storms	9	16	15	24	9	17	16	16	25	13	11	12
Mean precipitation	3.21	3.71	3.77	4.02	3.83	2.95	2.82	3.46	3.66	3.04	2.92	3.58

irrigation. As these storms are not so violent, or rather the precipitation is not so great, in June, July and August, the chances are somewhat more than even that they will prove a benefit, rather than a drawback, the mean precipitation only ranging from 2.82 to 3.46 in. for each storm, of which only sixteen or seventeen have occurred in 32 years, or one every other year in these months.

The rainfall, therefore, to be pumped off the land after it has served its purpose, is the mean of the monthly precipitation occurring during the rice-growing months, as shown in Table 11, plus the quantity put on the land monthly by siphons or pumping, plus the quantity added by infiltration, and minus the quantity lost by evaporation; and the pumps must be capable of taking this water off in a short time, say two or three days.

Table 10 shows the greatest monthly precipitation in each year, for the last 6 years, in the months from March to August, inclusive, in each of the eleven stations surrounding this tract.

Part of Table 10 is collated from the record of rainfall at New Orleans kept by the U. S. Weather Bureau, and part from the record of rainfall kept by the New Orleans Drainage Commission.*

These two records are interesting as showing the difference between two records kept on the same dates in different parts of the same city, and they emphasize the fact that the records of one station cannot be accepted as a basis for computations, but that the means of all cir-

**Journal of the Association of Engineering Societies*, Vol. XXVIII, p. 365.

cumjacent stations must be used to represent the actual precipitation on any tract of more than 1 mile square. They also make plain the absurdity of carrying out such computations to more than two places of decimals.

TABLE 10.—Greatest Monthly Precipitation, in Inches, in the Months from March to August, Inclusive, for the Past 6 Years, at Eleven Stations.

	Mar.	Apr.	May.	June.	July.	Aug.	Remarks.
	6.54	12.78	3.14	7.19	10.79	8.66	(Greatest in 32 years for New Orleans, the other for 6 years.)
	5.80	11.70	6.29	9.10	10.82	5.70	
	6.73	5.75	4.53	7.80	12.38	10.67	
	6.50	10.69	3.85	10.91	10.71	8.05	
	6.84	8.18	6.07	8.09	18.39	9.69	....
	6.09	12.07	4.30	7.78	14.30	8.50	....
	3.35	13.63	2.70	17.61	8.82	14.74	....
	11.32	13.78	18.68	8.01	9.87	22.74	....
	5.22	10.64	10.33	13.33	9.04	7.58	....
	.....	.....	.....	.....	8.75	.....	....
	.....	14.20	.....	8.04	6.20	.....	....
	.....	.....	.....	.....	8.96	.....	....
	.....	12.27	.....	12.05	10.71	.....	....
	.....	.....	.....	.....	18.39	.....	....
	.....	.....	.....	10.47	12.38	.....	....
	.....	.....	.....	.....	11.51	.....	....
	.....	.....	.....	4.70	9.61	.....	....
Means.	6.49	11.48	6.65	9.69	11.27	10.70	

Water Required.

Table 11, compiled from the records of the U. S. Weather Bureau, shows the monthly mean precipitation and the greatest monthly mean precipitation for each of the six rice-growing months, with the difference between them, and the quantity required for rice cultivation. This table shows that once every 6 years, on the average, there will be an excess of rainfall above the means for that month, and these months of larger precipitation may occur in groups or all in one year.

The mean excess, as shown in Table 11, amounts to 48 per cent. Table 11 shows that in some months of excessive rainfall, when water is required for rice, 4.92 in. more water falls than is required, which is 31.83 in., including what will be lost in evaporation, and adding 5% for infiltration. As the quantity lost by evaporation here is 4.17 in. more than the mean of Crowley and Texas, the quantity of water required here is increased that much more than the requirements in those places.

TABLE 11.—Greatest Monthly Mean Precipitation, in Inches, for Each of the Six Rice-Growing Months, etc.

Month.	Greatest monthly.	Monthly.	Difference.	Inches required.	Remarks.
March	6.49	3.68	2.81	Very little	(Means of 11 circumjacent stations for 6 years.)
April	11.43	5.35	6.08	" "	
May	6.65	2.24	4.41	" "	
Sums	24.57	11.27	13.30		
June	9.69	5.17	4.52	4.77	
July	11.27	6.83	4.44	17.98	
August	10.70	6.19	4.51	9.08	
Sums	31.66	18.19	13.47	31.83	
Means	9.37	4.91	4.46		

From the foregoing, Table 12 has been compiled, and shows the quantity required for the land, and which is supplied by rainfall, irrigation and seepage; also, the quantity lost by evaporation, leaving the remainder to be pumped off in those parts of this tract too low to drain by gravity at low tide.

TABLE 12.

Month.	Inches of Water Supplied by:			Inches of Water:		
	Rainfall.	Seepage.	Irrigation.	Required.	Lost by evaporation.	To be pumped off.
March	3.68	0.18	0	3.86	3.79	0.07
April	5.35	0.27	0	5.62	3.80	1.82
May	2.24	0.11	0	2.35	4.20	0.0
June	5.17	0.26	0	5.43	4.98	0.45
July	6.83	0.34	10.81	17.98	7.39	10.59
August	6.19	0.31	2.58	9.08	7.05	2.03
Totals	29.46	1.47	13.39	43.66	31.21	14.96

Table 12 shows that, generally, the rainfall and seepage or infiltration, in the vicinity of New Orleans, is enough, or more than enough, for the first four months of the rice crop, but that water must be supplied by irrigation in July and August, the quantity to be pumped off

afterward being about equal to that supplied by irrigation in these months. In March, April and June the evaporation is nearly equal to the rainfall, and in May nearly double. Comparing the evaporation with the quantity required and the greatest monthly precipitation, as given in Table 11, it will be seen that in months of greatest precipitation more than enough rain falls every month, except in July, when 6.37 in. have to be supplied by irrigation.

The greatest quantity to be pumped off in any one month is 10.55 in. in July. This would be equivalent to $43\,560 \times 0.88 \times 7.48 = 286\,729$ gal. per acre, and one Menge pump of the largest size would require 15 days to take this water off 4 sections, and 4 pumps would do it in 3¾ days, which is sufficiently rapid. The greater part of this land, however, is elevated 1 ft. or more above the Mean Gulf Level, and this would be sufficient to drain off by gravity, and the pumps would only be required to lower the water in the ditches. The pumps used for irrigating, in those places where siphons could not be used, could be used for drainage by a simple arrangement of valves and a by-pass.

The Menge pump may be described as an Archimedean screw, or perhaps as a turbine wheel, set in the water and run in the opposite direction to a wheel for power, or to a kind of submerged centrifugal pump, set with the shaft vertical in a square wooden box, and run with a belt from the engine. These pumps are used extensively in the vicinity of New Orleans for low lifts, and are much cheaper than even the centrifugal pump, which is cheaper than any of the others. The largest size Menge pump, 42 by 16 in., has a stated capacity of 2 000 000 gal. per hour, with a lift of 10 ft., or 33 333 gal. per minute, which equals 147.31 acre-feet per 24 hours, equal to 5 155.3 acres irrigated in 70 days, therefore eight of these pumps would be sufficient to irrigate four sections (allowing for land taken up by levees) 6 in. deep in 1 day, or the whole tract in 8 days, supposing the evaporation to equal the rainfall.

The price of a pump of this size is $937.50.

This computation is based on the rainfall and evaporation being the same for this land as at Crowley. This is not exactly correct, but it is not very different.

As stated before, every locality requires a separate study and computation based on the precipitation, evaporation, length of supply canal, etc., at that particular place.

The writer believes that the most economical plan will be to run all these pumps by electricity, an electric motor being placed in each station and connected with a central power-house. This will give a power ready to be used at a moment's notice; dispense with all the pump engineers and firemen except one each at the central station, who will run the whole plant; will save the expense of installing so many boilers and engines; and the expense of separate oil tanks and pipe line to each pumping station, or the greater expense of transporting fuel by lighters to these stations.

There is so little wood on this tract that this would be an expensive fuel.

Cost of Reclamation.

The cost of reclaiming the swamp lands, open and without trees or bushes, and supposing them to be laid out in 1-mile sections of 640 acres, and these sections grouped into one tract of 8 by 4 miles, will be as follows:

A protection levee, say 8 ft. high, with 4 ft. crown and 28 ft. base, and slopes of 1½ to 1, will be required all around the tract. This will be thrown up on the outside of the canal from which it is taken, and its size will determine the size of the canal, an allowance of 15% being made for shrinkage, which gives, for the size of the canal, 30.4 ft. wide on the top, 6.4 ft. wide at the bottom, 8 ft. deep, and with slopes of 1½ to 1. A head-gate will be built to connect this canal with the nearest navigable bayou or creek, to allow of its being navigated by boats.

A canal of smaller size will be built up along the center of the tract longitudinally and also one transversely, to allow of convenient access to different parts of the tract by boats. Smaller canals will be dug around each section and through the center, north and south, with ditches on intermediate lines, as shown by Fig. 1. The actual cost, if laid out on this plan, will be as given in Table 13.

If this land should be taken in single sections, and it should be necessary to bring the water in a long flume or ditch, and pump also, with a reservoir, the cost would amount to $16.50 per acre, while, on the other hand, in those locations where no pumping would be necessary, but the land could be flooded and drained by the action of the tide, as previously explained, the cost would be only $2.41 per acre. Each of these cases can occur on these lands.

TABLE 13.

Excavation, made into levees, 631 381.2 cu. yd. at $0.05	$31 569.06
42 by 16-in. Menge pumps, eight at $937	7 496.00
Eight pump houses, and setting up pumps	4 000.00
Electric motors (50 h.p.), eight at $500	4 000.00
Feed wire, No. 0 copper, say 25 miles	5 930.00
Poles, insulators and setting	1 500.00
Sluices set, 192 at $30	5 760.00
Two wooden head-gates, set up	4 000.00
Plowing, and burning levees	1 000.00
Surveying land and dividing into sections, 52 miles at $6	312.00
	$66 567.06
Contingencies, omissions and engineering, 10%	6 656.71
Total: 20 480 acres at $4.35 per acre	$73 223.77

As to the time required to accomplish the work in the case under consideration, which may be taken as a fair average condition, if the latest and largest improved suction dredges are used to dig the canal and make the levees, which they can do by using a plank form, which would be mounted on broad wheels and dragged along by using a snatch block and tackle attached to the engine of the dredge, all the canals of large size could be dug in a month after the machinery was on the ground, and the smaller canals and ditches in 4½ months, some of the latter being cut at the rate of 1 mile in 1½ days. Allowing for the time required to plow the levee bases, and for delays and stoppages incident to such work, it is safe to say that the 20 000 acres could be ditched and leveed ready for a crop of rice, with all gates set, in 9 months.

The cost of operating this plantation, outside of the farming operations, which would be done by the lessees, would be very little indeed, and would only be the cost of fuel and the wages of three or four pump men, and the cost of running an electric or alco-vapor launch, to be used by these men in visiting the pumps for the purpose of oiling and repairing them, and the windmills, etc., opening and closing the water gates and keeping the canals clear of weeds, total per year $20 000.

As to the income, the opinion is unanimous, among all in that district with whom the writer has conversed, that these reclaimed lands would rent easily and quickly for $5 per acre, up to $14, with water supplied, paid in rice.

TABLE 14.

20 480 acres at $5 per year		$102 400.00
Less cost of superintendence, fuel and operation	$20 000.00	
" commissions and advertising	10 000.00	
" interest on first cost, 10%	8 906.38	
		38 906.38
Net profit per year, $3.10 per acre		$63 493.62
Or, if planted in sugar, 50 cents per ton of cane raised, or $10 per acre		$204 800.00
Less cost of superintendence, interest, etc., as before		38 906.38
Net profit per year, $8.10 per acre		$165 893.62

If the land were planted with rice by a company, and the rice were sold to the millers, then, allowing a profit of 20% from the farming operations, which is certainly low, and taking the average production per acre at 14.2 bbl. for this region (the mean of several statements), the account would stand thus:

20 000 acres, at 14.2 bbl. per acre = 284 000 bbl. of rough rice at $3.50	$994 000
Net profit, 20% per year	198 800

As rice is not a cultivated crop, only requiring planting, watering and harvesting, and no fertilizers, the net profit thereon is much greater than on wheat or most other crops, and is nearer 40 than 20 per cent. It is highly probable, also, that these lands will produce as much, if not more, than those of Georgia and the Carolinas. Dr. Knapp states that the profit on rice growing is 55 per cent.*

The reports of the U. S. Department of Agriculture state that when the first large canal near Crowley, La., was completed and

* "Rice Culture in the United States," Farmer's Bulletin No. 110, U. S. Department of Agriculture.

operated successfully, the average price of rice lands rose rapidly from $7 to $10, $15, and $20 per acre.* Rice lands under the large canals around Crowley, La., are now held at an average price of $30 per acre, a few choice locations bringing $50 per acre in 1901.

These delta lands are so much richer than the prairie lands that, measured by their productiveness, they should easily bring twice as much, to say nothing of their immediate proximity to the metropolis of the State.

These lands are of the same fertile character as the rice lands of the Carolinas and Georgia. The former have produced 32 bbl. of rice per acre and more than 8 000 lb. of sugar. The prairie lands of Louisiana and Texas do not produce an average of more than 12 bbl. per acre.

These delta lands, as soon as they are relieved of water, can be leased to small farmers for from $5 to $6 per acre per year, or for 4 bbl. of rice where irrigation water is furnished and for 2 bbl. when the land is only drained. In some cases the leases, on such lands in this district as have been reclaimed, are for one-quarter of the crop, and, in this region, there is great demand for these reclaimed lands.

There is no doubt that they would be worth $200 to $300 per acre for truck farming when reclaimed, as some of these lands are only 3 miles from the center of the City of New Orleans, with a population of about 340 000.

* "Irrigation of Rice in the United States," 1902, Bulletin No. 113, p. 14.

DISCUSSION.

E. L. Corthell, M. Am. Soc. C. E. (by letter).—This paper is upon a subject which is not often brought before the Society, and yet it is a most interesting one. The general and detailed features have been covered very completely and ably by the author, and the paper will constitute a valuable source of reference for the members of the Society when considering at any time the subject treated. The writer desires only to give some additional information from his own experiences and study of the data upon the "Location and Character of the Lands of the Mississippi Delta." This feature of the subject is worthy of a paper of considerable length, but in this discussion only a general *résumé,* with some interesting facts, can be given. Mr. Corthell.

The area under consideration, and a much more extensive area, is practically a new country, certainly a new country geologically speaking, for it was made long after the geologic age and even within the life of man upon this earth, for it is hardly more than 4 000 years old.

In connection with the work at the mouth of the Mississippi River and the project for a bridge over that river immediately above New Orleans, certain facts from the borings made by the writer and from those made by others, as well as from a general examination of the whole area of the Mississippi Delta, enable us to understand something of the character of the material composing the delta and, perhaps, to arrive approximately at the depth of the same. Enough has been ascertained by borings and other examinations to show that the Mississippi River, in the delta proper, at least in the last 200 miles of its course, does not flow in "a channel belonging to the geologic epoch antecedent to the present," as claimed by a distinguished writer about half a century ago.

While investigating the bed of the river, to a depth of 200 ft. below low water, for the proposed bridge above New Orleans, the writer requested Mr. Frank T. Howard, of New Orleans, who had some years before sunk an artesian well in Lafayette Park in that city, to state what materials were passed through. The following quotations are from his letter:

"Drift formations, clay, white sand, marl, gray sand, yellow sand, mud, in different layers. Seven hundred and five feet, drift-wood pumped up.

"Seven hundred and forty-two feet, clean, coarse sand and gravel. The gravel became coarser as the drill descended, and rock was expected, but the drill suddenly entered a stratum of clay, broken shells and drift-wood, 69 ft. thick.

"Eight hundred and nineteen feet, fine blue sand, which contained sea-shells, coral and many kinds of fossils.

Mr. Corthell. "Eight hundred and fifty-three feet, clean white sand, salt water; at bottom of this bed large quantities of drift-wood came up, cedar and cypress woods.

"Eight hundred and seventy-nine feet, pipe slipped through large hole about 18 ft., and water was pumped down for half an hour, but none came up.

"Eight hundred and ninety-eight feet, bed of hard clay, 7 ft., under which was a quicksand, hard to penetrate.

"Through 108 ft. more, different layers, drift-wood, clay, and shell banks.

"Ten hundred and forty-two feet, *drift-wood.*"

Mr. Howard is of the opinion that that part of Louisiana was once covered by the Gulf of Mexico, and that the depth of water was probably 2 000 ft. or more. From some other information it is known that the drill broke, finally, in a cypress log.

At Magnolia, the plantation of Governor Warmouth, 45 miles below New Orleans, a well sunk to 980 ft. showed similar stratifications, though no wood was encountered. From the bottom of the well there came up, with very salt water, an inflammable gas, which, when set on fire, gave a fountain of flame.

Many other wells have been sunk to a depth of 400 ft. or more, all of them showing nothing but alluvium.

The investigations of the Mississippi River Commission, and the observations of many people competent to form a judgment, indicate that before the construction of levees, which prevent the overflow of the river, the annual accretions, deposited upon the older layers of material dropped in preceding years and centuries, brought additional weight upon all the delta lands, and in the lower part of the delta, nearer the Gulf, where no levees exist, this action is still going on. This weight produces a subsidence and compression of the lower strata, but, owing to the fact that the annual accumulation is slightly in excess of the subsidence, the latter action is not noticed, except in the area protected by the levees, where the overflow is prevented.

It was the opinion of the late Major C. W. Howell, M. Am. Soc. C. E., United States Engineer, who was for many years in charge of the improvements at the mouth of the Mississippi, that this annual superimposed weight upon the outer or Gulf slope of the delta caused the formation of mud lumps, which are found at the mouths of some of the passes of the river. The movement appeared in the form of ridges or wrinkles by which sometimes several acres of a sticky clay appeared in a few hours and sometimes rose to a height of 10 or 12 ft. above the water.

In reference to the important subject of the general subsidence of the delta, the writer is personally familiar with the following detail:

Mr. Corthell.

Several hundred years ago, the Spaniards built a magazine, of brick masonry plastered with mortar, on Balize Bayou leading out of one of the now unused passes of the river. While the writer was engaged in building the jetties at the mouth of the South Pass, he examined this old structure, which was in a good state of preservation. The exterior was intact, as far as could be seen and examined, there were no cracks in it indicating a settlement, and it was perfectly level, but the surface of the water was across the arch which crowned the entrance door, the sill of which was at least 10 ft. below the water. Twenty years afterward, the writer had this magazine again examined. It had gone down somewhat further, and enough of the roof and arches remained to show that the settlement had been proceeding at about the same rate as for 200 years previous, and averaged about $\frac{1}{20}$ ft. per annum.

Even at the Head of the Passes, where the ground is apparently solid, the same subsidence is going on. At the head of the South Pass, the rails of a railroad track, which had been laid many years before for carrying coal from the barges to the coal pile for the United States Government, were found to be projecting from the bank and 2 or 3 ft. below the surface of the ground, and it may be said that they were relatively below the surface of the water. These rails had not sunk into the material, for the ground was hard and solid, but they had evidently gone down with it; and, as this ground went down, the annual deposits from the overflow of the river have kept the surface at about the same level in respect to the level of the water surface.

Twelve miles above the head of the South Pass, or about 24 miles from the mouth of the river, it was found that a telegraph cable, which was put down many years before, and not more than 1 ft. below the surface in a small ditch when laid, was from 5 to 8 ft. below the surface, and drift-wood had to be cut away to reach it. In order to investigate a little further, a willow tree was taken for examination. This tree was about 10 in. in diameter, and was back of the shore near the edge of the swamp. A dam was made about the tree, and an excavation, which finally reached a depth of 8 ft., was dug on one side of it, and the tree showed rows of dead roots all the way down.

Two or three miles above the Head of the Passes, was built, many years ago, a pilot's house, on brick piers 5 ft. high. The ground is now up to the sills of the house, but the surface of the ground bears about the same relation to the water surface in the river as when the house was first built.

Land in the Lower Delta is not only going down, but there are lateral movements which are very interesting. The base line along a solid island at the mouth of the Southwest Pass, lengthened in a few years, by accurate measurements, from 700 to 712 ft.

Mr. Corthell. The writer was employed in 1895 by the estate of the late James B. Eads, M. Am. Soc. C. E., to collect data to present their case before the War Department. The case was briefly the following:

The contract with the Government, made by Congress in 1874, provided for the maintenance for 20 years of the channel to be obtained by the jetties; \$100 000 was to be paid quarterly as the estimated cost of maintenance; \$1 000 000 was held back as security for the maintenance of this channel, one-half of it to be paid after 10 years, and the other half after the remaining 10 years. This amount was assumed to have been earned, and the interest on it at 5% was to be paid semi-annually. The amount per day in the first 10 years was \$410, and \$324 during the second decade. The act required:

"The respective depths and widths of channel being measured at the average flood tide, as ascertained and determined by the Secretary of War."

"The average flood tide," as the datum plane, was established from the record of an automatic tide gauge, which ran continuously through three lunations, and read at average flood tide 2.76. It was established at Port Eads. It was not long before a slight discrepancy appeared, the mean of all the high waters of each year gradually reading higher on the gauge as the years went by, until, in 1890, the mean of the year read 4.001 ft. Either the gauge had gone down and all its reference bench-marks, one of which was on the foundation of the light-house not far away, or else the surface of the Gulf had risen progressively from 1875 to 1890. The writer, by correspondence with engineers at various ports on the Gulf, in the United States and in Mexico, ascertained that their tide gauges showed no change in the Gulf level during this period, and, after favorable consideration by the Secretary of War and a discussion of the legal questions, the datum plane was changed to conform to existing conditions.

Without going further into these details of subsidence, it may be stated generally that the question is one of great importance to the whole Delta country; that is, the outlying agricultural districts. There are really no compensating conditions, for the reason that the river, along its lower course in Louisiana, except near its mouth, is fully and safely leveed by the United States Government and the State of Louisiana. The late G. W. R. Bayley, M. Am. Soc. C. E., Mr. Eads' associate, a long-time resident of Louisiana, and engaged in many of its important works, was a very careful observer and an engineer of excellent judgment. He had wide experience with the levees of Louisiana, as well as upon the construction and maintenance of the New Orleans and Mobile Railroad. He was always very pronounced in his opinion that the subsidence of

Mr. Corthell.

the Gulf lands was going on, and was becoming more and more apparent as the river was becoming more and more leveed.

The subsidence of these lands is not peculiar to the Delta of the Mississippi, but the same action is taking place on all coasts where there are sedimentary deposits, such as Holland, where the rate of subsidence is about $\frac{1}{10}$ in. a year, but the rate of subsidence in Louisiana is considerably greater.

The lands which the author includes in his general discussion of rice lands and other lands, on which formerly the finest sugar cane was raised, are affected by this general subsidence. Many large tracts which were formerly very productive in rice and particularly in sugar, have been abandoned in consequence of the salt water encroaching upon them. The people believed that it was due to the rising of the Gulf waters, but it is really the effect of the sinking of the land. Many of these lands are now completely covered with sea water. Many islands on the Gulf Coast—Caillou, Last Island, and others—which were formerly pleasure resorts for the people of New Orleans, and were well above the water—have been abandoned for some time and are becoming marshes. Even in the interior, north of Lake Ponchartrain, there is proof of altered levels, although the change is less marked than on the coast. Regardless of this subsidence, the richness of the delta land and the practicability of protecting it from overflows of the river and encroachments of the sea, make the subject treated by the author of great importance to the State of Louisiana. With the modern methods of raising rice, and its increasing use, there is no doubt that these delta lands will produce an important and increasing product for the benefit of the entire country.

Mr. Le Conte.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This subject has always interested the writer, who has had considerable experience in work of this class. Of all enterprises, this is, without exception, the most alluring and the most fascinating. If a favorable location is selected, if the works are faithfully executed in a good workmanlike manner, if properly administered, and if judgment is exercised in keeping up all necessary repairs; then most wholesome and satisfactory results will follow, but not otherwise. The supervision of such works demands a high order of intelligence—too high for the average citizen—and, as a general rule, such enterprises too often fail from sheer neglect.

The writer is pleased with the way the author has handled the subject. The engineer in charge should always consider the worst case that could possibly happen, because the worst always happens. He mentions a Gulf storm raising the level of the water 7.0 ft. above mean low water, but does not state how long the water was maintained at that level; nor whether similar storms are prevalent

Mr. Le Conte. during the cropping season. It is true that in the case of rice culture the flooding of the tract by heavy rainfall, while the gates are closed by outside storm waters for two or three days, would do no serious damage, but, with other crops, it might be quite embarrassing, to say the least, and, if much protracted, might cause serious loss.

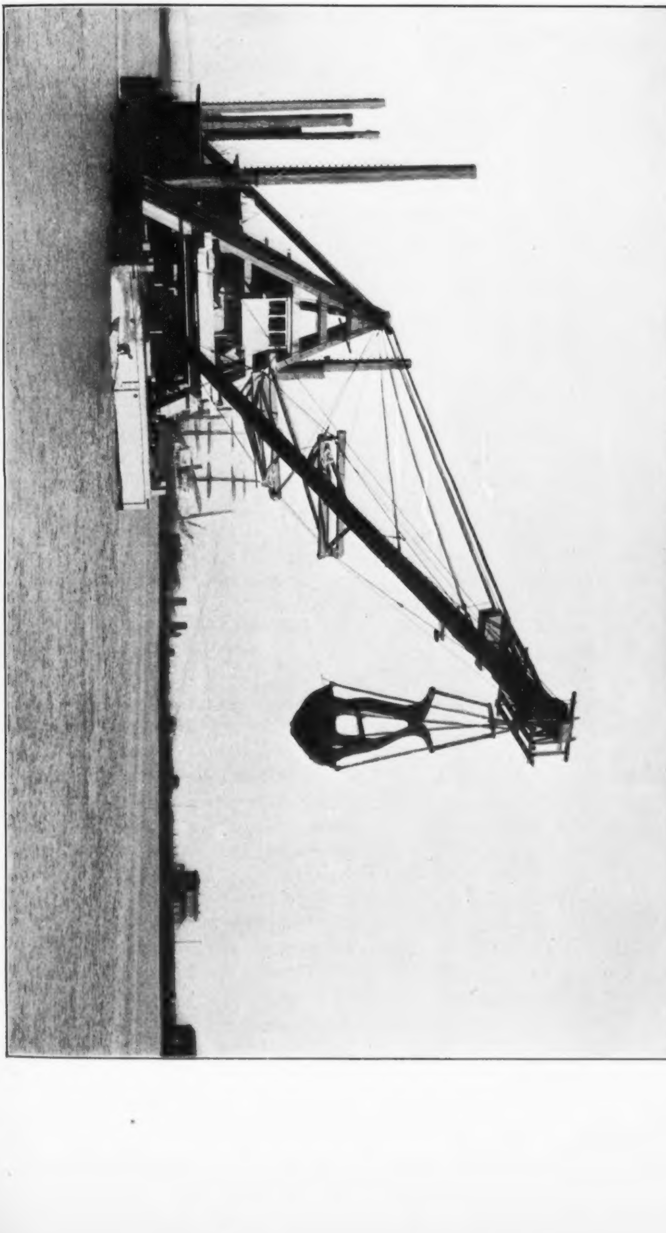
In laying out the lines of canals and ditches in the typical 4 by 8-mile tract, the writer was interested to note the apparent change from the old and well-established plan of such works, as generally laid out. According to the author's plan, the water surface in the canals and ditches, collectively, will amount to one-sixtieth of the total area of the tract; whereas, under the old general plan of canals and ditches, the water surface in all the canals and ditches combined would amount to one-seventy-fifth of the total area of the tract. This may be somewhat hypercritical, but, nevertheless, it is worthy of mention. It may be true that irrigation facilities may call for the change from the customary plan, and probably they do, to a certain extent.

The writer was surprised to note the small quantity of water needed for rice culture—29 and 37 in., including rainfall—and that the time extends only over from 64 to 70 days. This fact is of paramount importance, and puts new life into the business.

The writer has never seen the Menge pump, but can well imagine its good features. From long experience, however, he is inclined to prefer the plain centrifugal pump, as being best adapted to such work. The pumps of the best type are those having runners with a face width equal to the radius, so that the free openings in the runner have a width equal to the diameter of the suction pipe. Friction is very much reduced by this design, and, in the long run, it makes a marked difference in efficiency. The greatest monthly quantity to be pumped, mentioned by the author, seems to be well considered. It agrees very well with the old English and Dutch rules of $\frac{1}{4}$ to $\frac{3}{8}$ in. of rainfall per 24 hours.

The cost of excavating—5 cents per cu. yd.—is considerably greater than the actual cost of doing such work in California. For small jobs, under contract, this figure may be fair enough, but, for a job of considerable size, it would pay the land proprietor to build his own machine and do the work himself, at a cost not exceeding 2 cents per cu. yd. in place. The machine best adapted to do this work economically is the clam-shell dredge, with a 150-ft. boom, capable of making long reaches. Such a machine would not cost more than $40 000, and would only require a captain, a cook and a crew of four men on each shift. The daily expenses would not exceed $60, and the average output would be at least 3 000 cu. yd., making the actual cost not greater than 2 cents per cu. yd., as above

PLATE XI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 990.
LE CONTE ON
RECLAMATION OF RIVER DELTAS.



DREDGE WITH LONG BOOM AND "CLAM-SHELL" OF SPECIAL FORM.

mentioned. This dredge is shown in Plate XI. It will be noted that the bucket is of a type out of the ordinary, and is the result of long experience in this particular class of work. The efficiency and ease with which it can be handled cannot be praised too highly. The lever arms, extending back from the bucket on each side, answer two purposes, both of which are of inestimable value; namely, the whole weight of the bucket and leverage is available for closing, and, when opening the bucket, the weight of the arms exactly balances the weight of the shells; so that, figuratively speaking, a school boy could manipulate the empty bucket. The capacity of these buckets ranges from 3 to 10 cu. yd. The writer is inclined to think that for all practical purposes the 5-cu. yd. bucket is the handiest size. Mr. Le Conte.

RICHARD LAMB, M. AM. SOC. C. E.—The speaker would like to know whether the subsidence of the marsh lands on the deltas of the Mississippi are referred to by the author, also, whether the revetment described in the paper occasioned an accretion. It is an engineering fact, not generally known, but which was known to the Romans, as far back as the first centuries, that if a piece of marsh land on the delta of a tidal river is enclosed, there will be a natural accretion of land. Some years ago, on learning of the accretions which had been developed in England, and in making a thorough study of the subject, the speaker made some tests. A barrel was put in the sand in a river where the rise and fall of the tide was about 3 ft., and in a short time the sand came over the top of the barrel, although the general level of the sand was near the bottom of the barrel. Working on that principle, about 50 acres of land at tide water were reclaimed for the Norfolk Terminal Company. The natural accretion was augmented, however, by material excavated from the channel near by. Mr. Lamb.

The Romans, at the time of the invasion of Great Britain, reclaimed thousands of acres of land in "The Wash," on the north shores of Norfolk County. Three rivers, the Ouse, the Welland and the Witham, whose courses are very tortuous, empty into this bay. The channels of these rivers were straightened by levees on either side. The levees were built on the concave side first so that the water would scour out ahead of, and make a bed for, the levees. By straightening the courses of the rivers, the general elevation of the water was lowered, reclaiming thereby large areas of land. At the deltas of the rivers revetments were built about areas amounting, in all, to ½ mile by 10 miles. After these revetments were built, the accretions were very marked and steady. First a settlement of sand, then of warp, then a scrubby growth and at last a growth of valuable grasses. These lands were leased at £4 per acre, and much of it was sold at £80 per acre. It is used chiefly for pasture, but is considered the richest of the English agricultural districts.

Mr. Lamb. In 1883 the speaker was employed by The Dismal Swamp Land Company, composed of gentlemen, the heirs of George Washington, to make plans for the drainage of the Dismal Swamp. George Washington was granted 40 000 acres of the Dismal Swamp in consideration of his valuable services to his State and Country in the French War. He made surveys of these lands, some of the lines of which the speaker had the pleasure of re-running.

Incidentally, the speaker wishes to remark that, in a discussion as to how a tree grows—he contending that trees do not grow in height, but simply gain by accretion—he stated that in running the lines of George Washington's survey, he found a witness-tree on which the bark showed no marks. On cutting off the bark, three axe marks were found at the height at which a man would naturally have made them. If the tree had grown in height, those axe marks would have been up in the air 70 or 80 ft.

Grubbing a drained swamp land is the greatest expense of reclamation. Grubbing costs at least $30 per acre. The speaker found that after the swamp was drained and enclosed with a fence, by turning in a herd of cattle they grubbed up the land and fertilized it in a few years, and that all that farmers could do, at a cost of $30 per acre, was done by the cattle at practically no cost, when counting the sale of calves and beeves. It would be instructive to know the author's estimate of the cost of grubbing.

It is generally believed that the Dismal Swamp is an unhealthy morass. On the contrary, it is one of the healthiest districts in the State of Virginia. Before the war, when slaves were recuperating from malaria, they were often sent to the Dismal Swamp to cut shingles, in order to regain their health. The juniper water, which abounds everywhere, has the wonderful property of allowing nothing to rot in it. Water taken from the Dismal Swamp has been used for drinking purposes on men-of-war on a 3-year cruise without getting stale. The speaker has drunk juniper water taken from this swamp, and said to have been for 20 years in the tanks of the old man-of-war, *Constitution*.

It is generally supposed that the Swamp is a dead level, and Lyle, the great geologist, said that its submergence was due to the vast springs therein. After diligent search, the speaker never found one spring there. The Dismal Swamp is on a side hill, and at very few places is the water still. By observing the direction of flow of the water the summit levels of the divides were noted—the water flowing each way from the apex. These streams were followed, and thus the contours were determined and the Swamp divided into drainage areas. At first, before learning these facts, the speaker thought that the Swamp was so level that it would be necessary to use the Elkenton system, *i. e.*, to dig wells and border

them with dry rubble. These wells are dug through the impervious clays forming the bed of the water of the Swamp. After passing through these strata, quicksand is reached, at the level of tide water. Naturally, the water passing into these strata would lower the level of the Swamp. However, there is at least a fall of 2 ft. to the mile to the various points at tide-water level about the Dismal Swamp. Therefore the drainage becomes a simple matter. Mr. Lamb.

Reference is made in the paper to the value of rice culture on the reclaimed land. It may be of interest to state that George Washington, who has always been considered a man of great versatility, engaged in the rice business at a point farther north than rice has ever been raised in marsh lands. Of course, high-land rice is raised around there to-day, but swamp rice is not raised north of the southern part of North Carolina. The sluices and embankments of the Washington farm, known as "Paradise Farm," are still to be seen. Within a short distance of this farm, there is a fall of about 14 ft. Here was erected a mill for cleaning the rice and for other purposes. It is said that the rice crops of Paradise Farm were the envy of the farmers of that part of the State. Washington was called away from this profitable business to serve his country. Later, after retiring, in a letter to a friend, he said that it had been his ambition to drain the Dismal Swamp and to continue his operations in that district.

The cost of draining land, as given by the author, is practically the same as that estimated by the speaker for the work in the Dismal Swamp. The lands of this swamp which were drained yielded for years, as far as the speaker knows, and still yield, 100 bushels of corn per acre. They are considered the richest in the world. The only form of fertilizer used is a little lime applied when first reclaimed to "sweeten" the soil.

It is an interesting fact that the rice from Virginia, and also that from North Carolina, has been used almost exclusively to export for seed for rice culture in China and India; and, even to-day, nearly all the rice of North Carolina is exported for seed, and three-quarters of the rice used in the United States is imported, in spite of the fact that its importation is hazardous from the fact that if there is much movement of the cargo in the boat the husks make the rice too brittle to use. The rice is milled in the United States. This fact insures that the swamp lands of the delta of the Mississippi and many of the arid lands which the Government is now reclaiming will be utilized for rice culture, thus saving the large duty and the great risks of deterioration, and lessening the cost of what may become the staff of life of America as it is of Asiatic countries.

Since the war, rice culture has almost ceased in the South, as it was a business that meant death to hundreds of slaves, it being one

Mr. Lamb. of the most unhealthy employments, for the reason that in putting the water on the rice fields and taking off, malarial bacteria are generated. At one place the owner of a piece of property showed the speaker a small grave yard and said, "There is one million dollars of money buried in that little field—dead slaves." With modern machinery to do the plowing and planting, however, the labor difficulty is being overcome.

Mr. Le Baron. J. Francis Le Baron, M. Am. Soc. C. E. (by letter).—The subject referred to by Mr. Corthell, in connection with the Delta Lands of the Mississippi River, is interesting, and in the writer's report on "The Ship Canal from the Gulf of Mexico to the City of New Orleans," written at the same time as his paper on Delta Reclamation, this matter is discussed, in connection with the works of the canal, and also in connection with a large hotel on Grand Isle and an electric railroad thereto. For these reasons these additional data are welcome.

The writer's first suspicions as to this subsidence were aroused by observing that the parade ground in Fort Livingston, at the mouth of Grand Pass, Barataria Bay, which was built before the Civil War, was covered with 2 or 3 in. of water at every high tide. It was also noticed that the water of high tides completely surrounded the now abandoned light-house on the sand spit at the western end of the Island of Grand Terre; also, that the remains of the old "tabé" building, on the north side of Grand Isle, known as the "Pirate's House," was now surrounded at high tides. Another fact is that the graves near by are now covered with salt-water grasses. It did not appear probable that these conditions prevailed when the buildings and graves were made. The writer was also surprised to find, on comparing the numerous soundings taken on his surveys for the canals, railroads, and reclamations previously mentioned, that the water held persistently from a few inches to a little more than 1 ft. deeper than was shown on the charts of the United States Coast Survey, made in 1878, and also of the United States Engineers, made in 1881-82, and this, too, in quiet bays and dead bayous where no currents existed. The writer was not prepared for this, as he had expected to find these waterways growing gradually shoaler from the effect of vegetation and of wave action on the banks.

While studying the hydrographs of the Mississippi River, made in the office of the State Engineers of Louisiana, it was discovered that it was conceded that the city gauge, at the foot of Canal Street, had sunk 0.44 ft. This was discovered in 1892, the gauge having been used since 1885.

Looking still further for light, it was found that the United States Engineers had been studying this subject, and had reported,*

* Reports, Chief of Engineers, 1899 and 1900.

from the results of their levels, that Port Eads was sinking at the rate of 1.006 ft. in 17 years, but that New Orleans was stationary. This rate, 0.059 ft. per year, is remarkably close to the annual rate of sinking of the New Orleans City Gauge, *i. e.*, 0.063 ft., but it is possible that the pile to which the gauge was fastened went down independently of the ground in which it was driven. **Mr. Le Baron.**

The effect of this subsidence, if continued at the same rate, would be to submerge such of the protection levees as were near Port Eads in about 111 years. It would, at the same time, deepen the canals and ditches in that vicinity about 6.5 ft. in the same time, which would greatly exceed the annual deposits. As explained later, this ratio only applies to the vicinity of Port Eads, as at New Orleans it would be zero, and it is probable that the great weight of the jetties and buildings at Port Eads has caused that immediate vicinity to sink faster than elsewhere. The reclaimed land, of course, would sink at the same rate, but, at the same time, it would be raised annually by the sediment deposited from the irrigation water.

The proportion of sediment in the Mississippi River water is given by Humphreys and Abbot* as 1: 1 471 by weight, and 1: 3 396 by bulk, which is the mean of three determinations, at New Orleans, of yearly averages.

The increment of elevation due to this deposition, however, would be insignificant, amounting to only 0.05 ft. in 100 years, and, therefore, it can be disregarded.

In regard to the subsidence, however, it must be noted that the foregoing computations are based on the movement at Port Eads, and the United States Engineers state that no subsidence is in progress at New Orleans. In the absence of any intermediate definite observations, it can be assumed that the rate of depression is in direct proportion to the distance between these two points. As stated in the paper, some of this land is only 3 miles from New Orleans, and more than 90% of it is only half the distance to Port Eads; therefore, for an average area of this land, the subsidence would be only about one-fourth of the amounts here stated.

In regard to Mr. Le Conte's question, as to how long the water of the Gulf was maintained at the excessive elevation of 7.0 ft. above mean low water, and whether similar storms are prevalent during the cropping season, the records show that this elevation was not maintained more than one day.

By reference to Table 8, it will be seen that these excessive rises occur about once every year; or, taking the period from 1890 to 1903, there were four cases when there were two excessive rises in a year, and in one year (1897) three occurred. Five times during this

* Professional Papers, No. 13, Corps of Engineers, U. S. Army, Report on the Mississippi River, p. 146.

Mr. Le Baron. period the maximum elevation was maintained for 2 days and once for 3 days, and all these rises, except one, occurred during the rice-growing months.

The writer is glad to note that the actual cost of similar dredging in California is considerably less than 5 cents per cubic yard, because some engineers in New Orleans stated to the writer that they considered 10 cents per cubic yard about right for the levees.

The average cost of back levees (such as these), in this locality, is from 8 to 10 cents per cubic yard, and these are the prices generally estimated by the Board of State Engineers. Where the work is on as large a scale as planned in the paper, however, it is believed that it could be done for much less.

Dredges in this territory have dug canals at an actual cost of $0.023 per cubic yard, their capacity being 1 200 cu. yd. per day. These were of the Menge, endless-chain, bucket type.

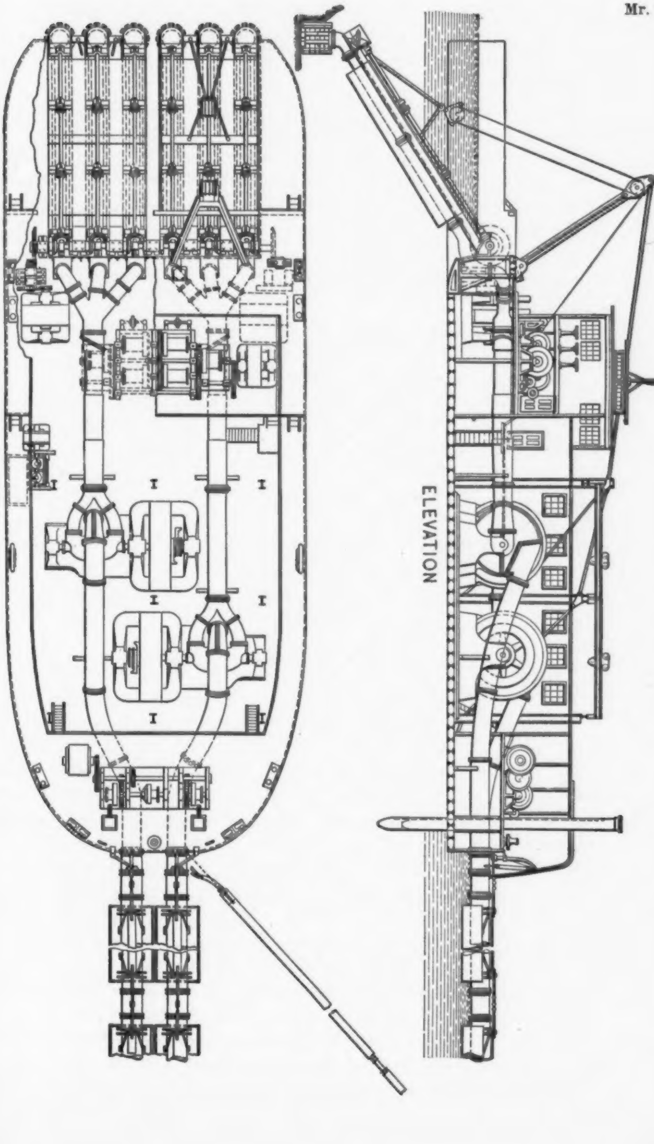
The dredging done in Florida, by the Okeechobee Drainage Company, of which the writer was Assistant Engineer, with continuous-bucket dredges, in sand and peaty muck, cost 3 cents per cubic yard, and the cost of dredging phosphate gravel in Florida, with a 9-in. centrifugal pump, was the same, the writer being the original discoverer of these phosphate deposits, and the first to report on dredging them, and having had the satisfaction of seeing his suggestions carried out, with the foregoing result.

Making a great jump, now, from the common sand sucker to the mammoth suction dredges invented by Mr. Lindon W. Bates, the average cost of dredging Mississippi River sand and mud was found, by tests of four different dredges of this type, to be $0.0097 per cubic yard, while running,* and the average quantity moved per hour was 1 056 cu. yd. Later work, done by Mr. Bates' large dredge on the Volga, in Russia, in river sand and clay, gave a capacity of 3 175 cu. yd. per hour, taking an average of seven trials.

It is well understood that this rate cannot be continued unceasingly, as time is unavoidably lost by moving into position, machinery breaks and requires repairs entailing stoppages, boilers have to be blown out, storms may stop work, and various accidents may, and some are sure to, happen, during which time the pay of the crew is going on and, perhaps, the coal is being burned. The writer has had considerable experience in work of this kind, and, in his opinion, the material to be dredged in the Mississippi Delta is probably the best that exists anywhere for excavating with these hydraulic dredges, as it is a mixture of clay, silt and sand, in varying proportions, the silt predominating, and being of about the consistency of mush. It is much better material than that in which the test before mentioned was made. In this material dredges of this type,

* Report, Chief of Engineers, U. S. Army, 1898.

Mr. Le Baron.



PLAN

Fig. 8.

3 000-H. P. ELECTRO-HYDRAULIC DREDGE

Mr. Le Baron. of the largest size, with proper management, can be depended on to excavate 500 000 cu. yd. per month, working day and night, which would be considerably less than half of the test capacity of the Volga dredge.

In this locality, everything is favorable for rapid work. The waters are completely land-locked, and the severest storms would scarcely interfere with the work; there are no vessels passing; no drift logs or flood currents; no ice; no rock or indurated material; no gravel, and so little sand that the abrasion will be reduced to a minimum. With the exception of an occasional sunken log, there is nothing to interfere with theoretical perfection in the working conditions, and the excavating units can be greatly multiplied on each pontoon.

This remark is intended to apply to the treeless portion, which forms about 75% of the whole area.

For these reasons, the writer must differ from Mr. Le Conte as to the style of dredge most appropriate for the work, as this is emphatically some type of suction dredge, such as shown in Fig. 3. For the small wooded portion of this area, however, a single-bucket, boom dredge, or the clam-shell, or orange-peel dredge, such as advised by Mr. Le Conte, would be desirable to pull up stumps and take off the top layer down to the limit of the tree roots. For cutting the ditches, a regular ditch digger would be required.

One of these large suction dredges will excavate fully six times as much per month, in this material, as the dredge mentioned by Mr. Le Conte. The first cost will be about three times as much, and the daily cost of operating five times as much. The cost of operating can be reduced nearly 50% by using electricity in this locality, as a great deal of money will be saved on the fuel. But it is one thing to excavate canals, and waste the mud, and another to make canals and levees. The levee base has to be plowed, and forms of some kind must be used to hold up the material excavated until it has time to solidify. For this purpose, traveling forms can be used, as stated in the paper, or brush and hay barriers can be erected. These strain out the water and leave the solid material. The levees can be made water-tight by building them of layers of mud and hay, the thickness of the former being about 3 in. and of the latter ½ in. when compressed. Built up in this way, the slopes may be as steep as 1 to 1, or even steeper. The hay can be cut on the spot. Taking all the foregoing matters into consideration, while there is no doubt that the canals can be excavated for less than 2 cents per cu. yd. with suction dredges, the writer's estimate of 5 cents for levees may be considered conservative and safe, but he would not want to take the contract for 2 cents.

In places where it is desired to underdrain with tile, as would be

the case where vegetables were to be raised, drainage machines should be used to cut the trenches for the tile. These machines will cut a trench from 3 to 5 ft. deep at the rate of from 3 to 5 lin. ft. per minute. Mr. Le Baron.

Mr. Lamb's remarks are very interesting, and the writer has found the same deceptive appearance in Florida. There are large tracts of swampy land between Jacksonville and Baldwin which appear to the eye to be perfectly level, but the writer found, when making surveys for the Florida Ship Canal, that these lands really rose toward the west at the rate of 9 ft. to the mile, in places, for several miles. Okefinokee Swamp, in Georgia, of which mention is made later, has a slope of 1 ft. to the mile, the dip being toward the west.

The revetment never causes an accretion, because the levees are never allowed to be overflowed.

In regard to grubbing, most of these lands are open marsh, covered only with grass. Taking an average, therefore, of the treeless and timbered land, it was thought that the marketable timber and fuel on the whole would more than pay the cost of clearing and grubbing. The cypress timber on these lands is valued at $100 per acre. In the low hammock lands of Florida, which are just too high to be classed as swamp, but, however, are inundated occasionally and are covered with a thick growth of large, deciduous trees, clearing and grubbing costs $100 per acre, grading down from this to about $25 per acre on open pine land.

As to grubbing up such swamp lands by simply turning in a herd of cattle, as Mr. Lamb states, the writer cannot understand this at all. The roots in the ground nearly equal the wood above. The writer has seen numerous pine-land lots "cow-penned," as the farmers in the South call it, by confining cattle in them. This is a very successful method of fertilizing, and an "institution for the promotion of laziness," but the roots remain alive in the ground, and will sprout and grow as soon as the cattle are removed. There is a vast difference between grubbing and clearing. Where land is covered with small bushes only, there is no doubt that goats will clear it perfectly, and if the goats are kept in it long enough they will eventually kill the brush growth by continually browsing off the sprouts, and the roots will then die.

It is stated by the United States Department of Agriculture* that "100 Angoras to 40 acres of brush land will clear it clean as a lawn and as perfectly set in grass, in two years." The common goat will do as well.† It is stated in the former Bulletin that the Angora goats cleared the land better even than Chinamen paid at the rate of $20 per acre. Of course, the large trees have to be cut down and

* Farmers' Bulletin No. 137; "The Angora Goat," pp. 12, 15.

† Circular No. 42, p. 12, Bureau of Animal Industry, U. S. Dept. of Agriculture.

Mr. Le Baron. the stumps pulled. On the Indian River Railroad, in Florida, the writer made a contract to have the stumps removed from the road-bed for 5 cents apiece. These were large hard pine stumps, and very full of pitch. The process consisted simply in building a fire against them and burning them up. With a little help in clearing away the earth around them, it was easy to burn them out for a depth of 3 ft. In regard to the fertility of these drained marshes and swamps, after the land has been "sweetened" by lime applications or by weathering, the writer fully agrees with Mr. Lamb. There is, in fact, but one opinion about it.

As stated in the conclusion of the paper, these reclaimed lands in the Carolinas and in Georgia produce enormous crops of rice and sugar. Dr. Hilgard, the eminent geologist, speaks of this region as "The most fertile agricultural lands of the State, equaled by few and surpassed by none in the world in productive capacity." Professor William C. Stubbs, Director of the Louisiana Agricultural Experiment Stations, says* that "over 100 bushels of oats, per acre, and the same of corn, have been grown on the alluvial lands."

"Forty tons of sugar-cane per acre is not unusual on these lands," and 44.02 tons have been raised at the Louisiana Sugar Experiment Station.† The present price of this cane is from $3 to $3.50 per ton at the mill (1903). The price paid is from 85 cents to $1 per ton for each cent per pound that "prime yellow sugar" brings in the New Orleans market. By way of comparison, it may be stated that on the reclaimed lands of the Disston Company, in Florida, of which the writer was Assistant Engineer, 6 210 lb. of sugar (30 tons of cane) per acre were produced. From 35 to 40 tons per acre are produced in St. Mary's Parish, La. These Florida lands also raised 80 bushels of corn per acre, and bounteous crops of watermelons and garden vegetables, which could be put in the market in March.

Mr. Columbus Allen, a well-known gentleman of New Orleans, says:

"These fresh-water marsh lands have long been known to be of wonderful fertility, specimens of their soil having long since been analyzed by Professors Jackson, of Boston, and Forshee and Riddell, of the Louisiana University, and were found by those eminent scientists to possess, in the most remarkable degree, every element of fertility and productive power, not only for cultivation, but for use as a fertilizer of most admirable character for the thinner soils of the uplands. The basis of the soil of the 'fresh-water marsh lands' is a hard blue clay, perfectly firm and of a character almost identical with that which is found at the bottom of the Mississippi River. Upon this basis, or substratum, there rests a black loam of light vegetable mould, varying in depth from 2 to 6 ft., which, when

* "Handbook of Louisiana," p. 31.

† "Sugar Cane," by Wm. C. Stubbs, A.M., Ph.D., Vol. 1, p. 49.

drained, will yield easily to the plow, and in fertility will prove equal to any lands in the known world. These lands, in their natural state, are covered with a dense growth of grasses and reeds, which spring up to a height of from 4 to 7 ft., and also the wild pea and potato vines, black and dewberry briars, and innumerable varieties of wild flowers of wonderful hue and luxuriance. Mr. Le Baron.

"Experiments made upon small portions of these lands, that have been reclaimed, prove them to be remarkable in their fertility and productive energies. Orange trees grow to great size, and their fruit is superior to that produced in any other portion of the State; and it is conceded that the Louisiana orange has no superior in the world. Figs, bananas, and, indeed, almost every other tropical fruit will grow there in great profusion. The sugar canes grown there are three times the size of those grown on the best sugar lands in the State."

Mr. M. W. Darton, Civil Engineer, who examined these lands in 1888, says, in a report made to some gentlemen of New Orleans:

"The marsh is reclaimable. The land is very rich when once placed under cultivation. The alluvial land is as rich as any in the world. It will produce in perfection all crops known to this latitude, as well as oranges, bananas and other tropical fruit. Terrebonne (Parish) is well adapted to orange culture. The climate of the Parish is excellent, freezes being nearly unknown and occurring only during severe winters."

Professor Stubbs says:*

"The rich alluvial lands of the southern part of the State will grow fine, thrifty orange trees without fertilization. Immediately on the Gulf Coast, anywhere from the Sabine to the Pearl River, all the varieties of oranges can be successfully grown. At present the chief locations of extensive groves are on the Mississippi River below New Orleans."

Dr. W. C. Stubbs, Director of all the Agricultural Experiment Stations of Louisiana, before quoted, speaking of these alluvial lands of the State, says:

"This region occupies about 19 000 sq. miles, and its vast possibilities in the near future for supporting millions of beings are simply inconceivable. The lands of this section are now leveed against the annual encroaching floods of the rivers which traverse them. Several millions of dollars are annually spent in enlarging and strengthening these protecting earth walls. When these streams, as they will be in a few years, shall be safely controlled in their annual rises, and the confidence of the people is established in the ability of levees to protect thoroughly, then will a full appreciation of the intrinsic merits of these lands be realized and high values be established."

The United States is now assisting the State in this levee work, and is annually expending large sums on the levees, which is a guaranty that they will soon be properly completed and maintained.

* "Handbook of Louisiana," p. 38.

Mr. Le Baron. As to the practicability of reclamation, there can be no doubt, for the reclaimed lands are seen everywhere in Louisiana, bearing enormous crops of rice, sugar, vegetables and fruit. One has only to ride over the New Orleans, Ft. Jackson and Grand Isle Railroad to Buras, or over the Southern Pacific Railway to Baldwin and Cypremort, to be convinced, not only of the practicability of reclamation, but of the unexcelled fertility of the land after it is reclaimed, and the comparative ease of accomplishing it. Both these railroads are bordered for hundreds of miles with these leveed and reclaimed lands, in a high state of tillage. The Louisiana Land Reclamation Company has reclaimed 13 000 acres.

The lands of the great Okefinokee Swamp, in Georgia, more than 675 sq. miles in area, which have been reclaimed according to the writer's plans, by a company of which he was Consulting Engineer, have produced enormous specimens of different crops, from 40-lb. watermelons to oats 6 ft. in height and heavily headed, and nutritious grasses 7 ft. high.

The Sarasota Saw-Grass Pond (500 acres), in Manatee County, Fla., which the writer examined and reported on for the projectors, cost, including all clearing, small and large ditches, etc., $6.50 per acre, all done by hand labor. The soil was a black, mucky peat, growing finer and more decomposed in the deeper parts. The top was quite light and porous. It produced in great perfection all kinds of vegetables. The water-table, at the time of the writer's visit, was only 2 ft. below the surface. The surrounding country had been suffering from a drought, but here water could be squeezed out of the soil at a depth of 6 in. It was found necessary to sweeten the land with lime at first.

The subject of health, in connection with residence on rice lands, has been mentioned by Mr. Lamb. It is generally supposed that all reclaimed swamps are deadly while the process of reclamation is going on. This was freely predicted in Florida when the work of the Okeechobee Drainage Company, previously referred to, was started. The result, however, proved exactly the opposite. No negroes were employed on the dredges, and most of the crew were from the North, but there was an entire absence of any climatic sickness among the workmen, most of whom lived more than two years continually on the dredge, in swamps so large that the high land could not be seen on any side. The same thing was predicted on the Nicaragua Canal work, of which the writer was in charge, but the predictions proved false. There was almost no climatic sickness.

In regard to the upward movement of blazes on trees, referred to by Mr. Lamb, the writer personally confirmed these observations

Mr. Le Baron.

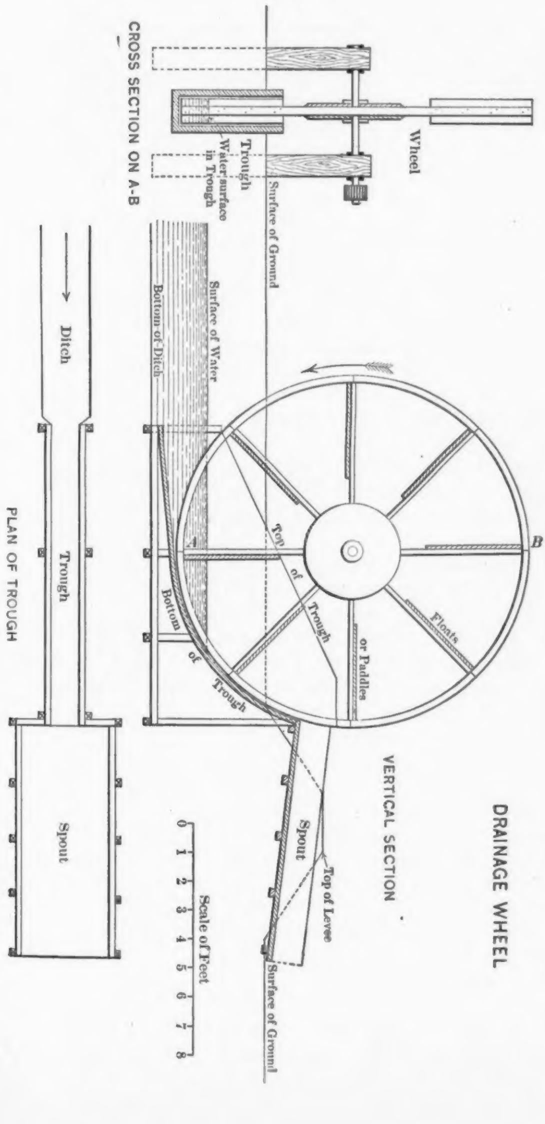
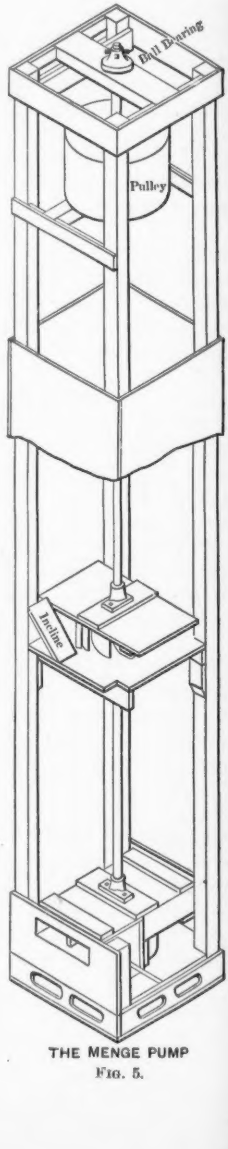


Fig. 4.

Mr. Le Baron. in fully a thousand cases in Florida, when he was a Deputy United States Land and Mineral Surveyor. The Public Land Surveys were made there from 57 to 70 years ago, and every section-corner witness-tree is a living witness to the fact that blazes are stationary. A case occurred in the writer's academy days. A certain land owner, in the Town of Groton, Mass., brought suit against a mill company for flooding his land above the authorized height. The professor of civil engineering in the academy was employed to ascertain if the dam was higher than the charter authorized. The decision hinged on whether the old bench cut on a near-by tree, which was used by the professor to ascertain the height of the dam, had moved up with the growth of the tree. The professor contended that it had not, and his contention was sustained by the court.



THE MENGE PUMP
FIG. 5.

Fig. 4 represents a wheel, such as is now used in the vicinity of New Orleans, to raise water when the lift is only a few feet. These wheels are usually about 10 or 15 ft. in diameter. They are easily made of wood by any local carpenter, and are run by a windmill or by a steam engine. They are cheap and very efficacious for low lifts, with practically no parts to get out of order. A deep wooden trough is set in the ditch, the bottom being concentric with the wheel. The latter pushes the water before it by the impact of the paddles, which fit close in the trough. In the drawing the lift is about 3 ft., but such wheels are not generally used for lifts of more than 2 ft.

Fig. 5 represents a double-lift Menge pump, used for raising water as high as 27 ft The single-lift pump is the more common

Mr. Le Baron.

type, and is all that would be required in delta reclamation work. The water enters the box at the bottom, through the numerous ports, and is forced upward by the revolution of the wheel set in the bottom of the box. The box is of wood, and the casing can be cut open on any side at any height desired and a spout set in. There are no valves. The casing shown above the upper pump is continued to the bottom. The writer has never seen an efficiency curve for this pump, or for the drainage wheel.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 991.

METHODS OF LOCATION ON THE CHOCTAW, OKLAHOMA AND GULF RAILROAD.*

By F. Lavis, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. E. Sherman Gould, Wilford A. Thompson, S. Whinery, C. P. Howard, Emile Low, F. T. Oakley, O. H. Tripp and F. Lavis.

Much has been written on the theory of railroad location, but the writer recalls little on the actual practice and modern methods of procedure in the field, and while he feels that there are many engineers far better qualified than he to take up the subject, the fact remains that they have not done so, and there is little public record of current practice. It is hoped that this paper will evoke some discussion, which will cover the subject from the many different standpoints and form a basis for more uniform methods.

The writer has been connected with surveys for the location of railroads in many parts of the United States and in South and Central America, during the past fifteen years, and has been impressed by the wide variation in methods adopted by different men and railroad companies, and, to a great extent, by their general assurance that the methods of each were right, and, having had all kinds of assistance and equipment, from almost nothing up, was, therefore, all the more

* Presented at the meeting of January 4th, 1905.

ready to appreciate the conditions under which the surveys were conducted which form the basis of this paper.

During 1902 the writer was engaged in making location surveys for the Choctaw, Oklahoma and Gulf Railroad (now part of the Rock Island System) in Oklahoma, Indian Territory and Northern Texas. F. A. Molitor, M. Am. Soc. C. E., was Chief Engineer, and E. J. Beard, M. Am. Soc. C. E., Principal Assistant Engineer, and to them the writer is largely indebted for much of the matter contained in this paper.

Many different lines were investigated, and, between 1898 and 1902, some 800 miles of branches and extensions of this road were built, scattered through five different states and territories. At times as many as ten or more locating parties were in the field at the same time; the methods adopted, therefore, were such as were adapted to maintaining parties in the field continuously.

The writer has confined himself entirely to the methods of making surveys, and the organization and equipment of parties, as practiced generally on this road, with whatever added notes from his own experience he has thought useful, and has not attempted to consider in any way the theory of railroad location.

It is obvious, of course, that the equipment of field parties must vary with the locality in which they are engaged. It is the general practice, however, except in a very few of the Northeastern States, to provide the parties with a camp outfit more or less complete, according to the policy of the road, its financial status, and the facilities of transport. It is the writer's experience that the completeness of the survey is very apt to vary directly with the completeness of the outfit supplied by the railroad, and he has, therefore, entered more or less minutely into details of camp equipment; for, although this has been noticed elsewhere, particularly in "Rules for Locating Engineers on the Northern Pacific Railroad," by E. H. McHenry, M. Am. Soc. C. E., the practice varies considerably.

The following is a list of the camp equipment furnished by the Choctaw, Oklahoma and Gulf Railroad:

1 Office tent with fly.......... 14 by 16 ft.
3 Tents........................ 14 by 16 ft.
1 Cook tent.................. 16 by 20 ft.
3 Drafting and office tables.
½ dozen camp chairs.
Stationery and map chest with necessary stationery, blank forms, drawing paper, etc.

Dining Table:

8 dozen agate-ware dinner plates.
8 " " " cups.
2 " " " saucers.
2½ " steel knives.
2½ " " forks.
2½ " German silver teaspoons.
1½ " " " dessert spoons.
1 " " " tablespoons.
½ " tin salt boxes.
½ dozen tin pepper boxes.
¼ " round agate-ware pans. 2 qt.
½ " " " " " 1 "
1 " " " " " 1 pt.
1 carving knife and fork.
7 yds. oilcloth, 48 in. wide.
3 standard trestles (see sketch, Fig. 3).
5 boards, 12 by 1½ in. by 18 ft. (dressed).

Cooking Utensils:

1 No. 8, 6-hole, wrought-iron range.
1 tea-kettle.
1 large cast-iron pot.
1 small " " "
2 large frying pans.
1 small " pan.
2 griddles.
4 tin pans with covers, 1 gal. each.
2 stewpans.
1 3-gal. coffee-pot.
1 gal. teapot.
4 dripping-pans.
6 baking tins for bread.
12 tin pie plates.
2 butcher knives.
1 steel.
2 large meat forks.
1 chopping-knife.
1 meat saw.
2 large iron spoons.
1 soup-ladle.
1 cake turner.
1 flour sieve.
1 colander.
1 5-gal. tin dishpan.
1 5-gal. " bread pan with cover.
1 chopping-bowl.
1 bread board.
1 rolling-pin.
1 biscuit cutter.
1 nutmeg grater.
1 coffee-mill.
1 spring balance.
6 galvanized-iron buckets.
6 tin dippers (one for each tent and two in cook tent).
2 can openers.
1 corkscrew.
1 broom.
1 scrubbing-brush.
1 alarm clock.
1 table (same as drafting tables).

Miscellaneous:

½ dozen Dietz lanterns.
3 large tin lamps (central-draft, round wicks).
2 large galvanized-iron washtubs.
1 washboard.
4 Sibley stoves (4 lengths of pipe with dampers, 12 lengths of plain pipe) (see Fig. 1).
2 water kegs, 2 gal. each.
6 washbasins.

Tools:

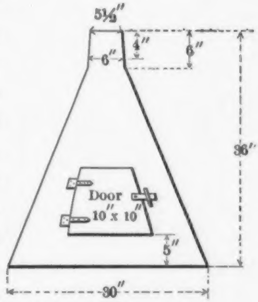
1 grindstone and fittings.
1 monkey wrench.
1 pick.
2 shovels.
1 short crowbar.
1 hand-saw.
1 cross-cut saw.
2 hand-axes.
4 chopping-axes.
½ dozen axe handles.
1 bundle sail twine.
½ dozen sail needles.
1 sail palm.
10 lb. assorted sizes wire nails.
100 ft. manila rope, ¾-in.

Lunch Box:* (See Sketch, Fig. 5).

2 dozen agate-ware dinner plates.
2 " " " saucers.
1½ " steel knives.
1½ " " forks.
1½ dozen German silver teaspoons.
1½ " " " dessert spoons.
1 2-gal. coffee-pot.

* On the Choctaw, Oklahoma and Gulf Railroad, this extra equipment for the lunch box was not ordinarily furnished; the writer, as explained later, believes it to be economy, however, to provide this.

Tents.—The tents furnished were of 12-oz. duck, roped on the seams and ridges with ¾-in. Manila rope. They were without ridge poles, four upright poles supporting the center, and four on each side supporting the walls. Tackle was, also, provided, and two single blocks on the front guy rope, there being rectangular door flaps at each end, with substantial leather buckles for fastening them. Leather stove pipe holes, with asbestos filling between the leather, were provided. The office tent had 5-ft. walls; the others, 4-ft.



SIBLEY STOVE.
No. 18 U.S.S. (0.05")Sheet Steel.
5 Joints Stove Pipe.
Damper in 1st Joint.
FIG. 1.

The writer believes that, when the genuine Mt. Vernon army duck can be obtained, 10-oz. duck gives practically as good service, as far as life is concerned, as 12-oz.; the stiffer duck, when folded, easily cuts and wears in the creases when carried in the wagons. In very hot weather, or in a very rainy country, a 12-oz. fly is desirable. In cold weather, with the lighter tents, in sizes above 14 by 16 ft., it is difficult to heat a tent of 10-oz. duck with the ordinary Sibley stove, but, if necessary to provide camp equipment in a cold climate, the whole equipment can be kept down to that size.

Drafting Tables.—The tops of the drafting tables (Fig. 2) were of ⅞-in. clear white pine, with hinged legs, connected by 3-in. webbing, arranged so that the legs folded flat against the tops. When moving camp, the tables were placed face to face and tied together, thus preventing injury to the tops.

Dining Table.—The planks forming the top and seats of the dining table (Fig. 3) are placed in the bottom of the wagon when moving camp, as they take up very little room in the bed of the wagon, and the projection of the planks at the rear provides support for the stacked Sibley stoves and other light equipment. The legs of the lower portion of the horses are so spaced as to straddle the wagon and drop down between the bed and the rear wheels.

Stationery and Map Chest.—It is important that this chest (Fig. 4) should be well and strongly made. The protection of the maps, etc.,

often the results of the expenditure of thousands of dollars, should not depend on any cheap or temporary expedient, as is often the case. It is not an uncommon experience to be caught in the rain while moving camp, and the best of tents are not always impervious to water. It should be insisted upon that all maps, notebooks, etc., should be placed in the chest over night. Necessary stationery, drawing paper, supplies, etc., will vary with different requirements and individual

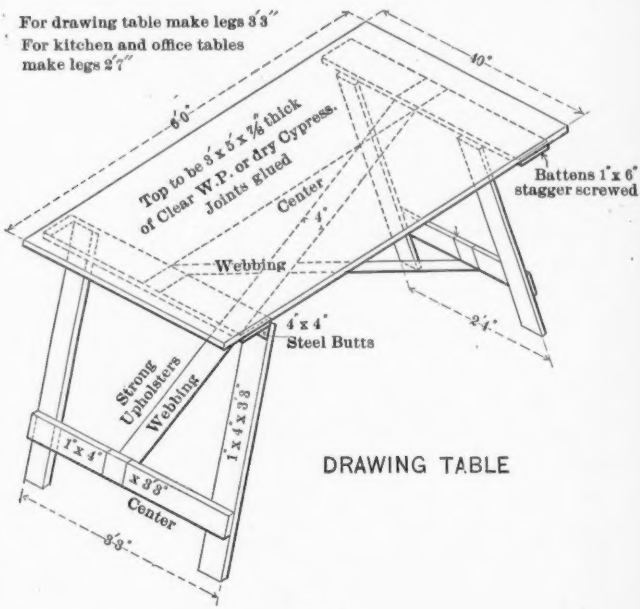


FIG. 2.

preferences. The list given by Mr. McHenry, in the book referred to, is quite complete.

Lunch Box.—The midday meal being eaten in the field, a substantial lunch box (Fig. 5) should be provided, with a separate equipment of plates, knives, forks, etc., from that used in camp.

The party should be ready to start for work immediately after breakfast, and should not be kept waiting while the breakfast dishes are being washed to go into the lunch box, nor should they have to

wait for their supper, while the dishes used on the line during the day are being prepared for use. The lunch box can often be best designed in camp after starting, so that it can be made to fit the supplies purchased.

On being organized, the party with which the writer was connected proceeded by rail to the point nearest the proposed line, at which place teams had already been engaged. The cooking utensils and a preliminary bill of supplies had to be purchased, however, and that and the loading of the teams occupied the remainder of the day of arrival.

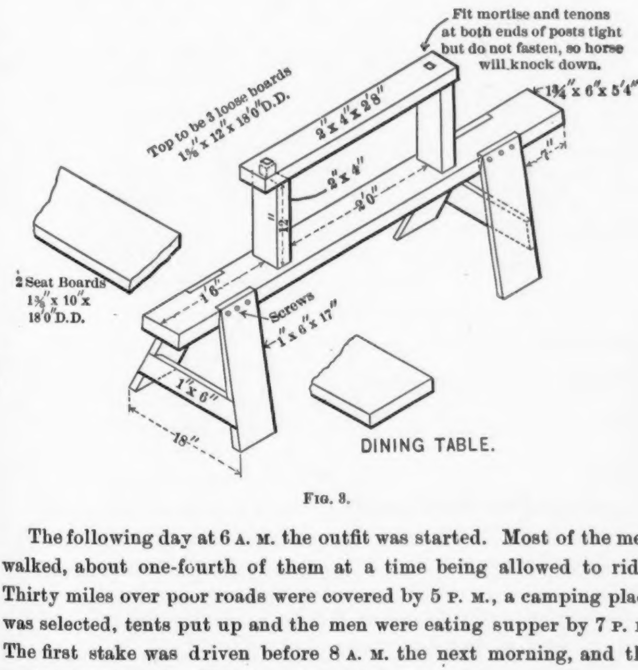


FIG. 8.

The following day at 6 A. M. the outfit was started. Most of the men walked, about one-fourth of them at a time being allowed to ride. Thirty miles over poor roads were covered by 5 P. M., a camping place was selected, tents put up and the men were eating supper by 7 P. M. The first stake was driven before 8 A. M. the next morning, and the work fairly started. This is not noted as an uncommon occurrence, but as representative practice.

In winter the men were called by the cook at 6 A. M. Breakfast was ready at 6.30, and a start was made for the work at 7 A. M.; in

summer, half an hour earlier. The teamsters were called and had their breakfast half an hour before the other men, so that they had their teams hitched up and ready to start as soon as the men had breakfasted. Two teams were used on the line, one staying with the topographers. The third team was busily engaged in keeping up subsistence, supplies, fuel, etc. It has been found more economical, and generally as satisfactory, to employ constantly only two teams in

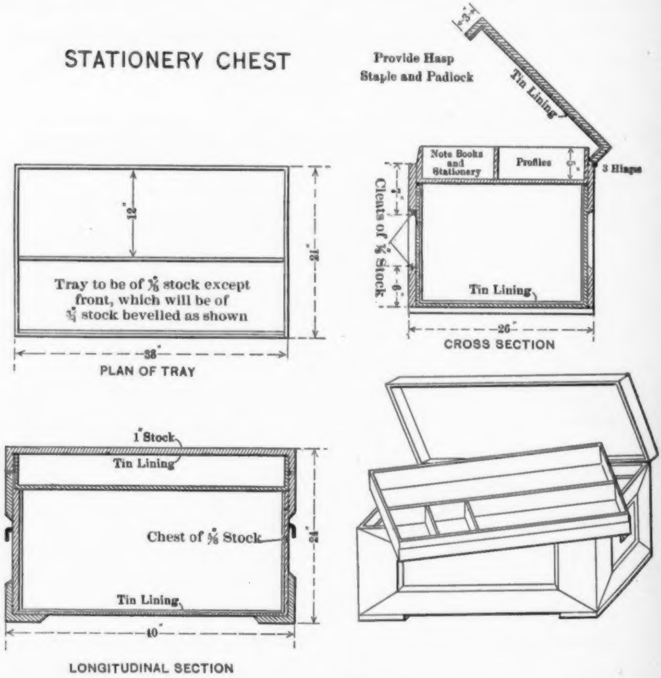


FIG. 4.

settled country where supplies can be easily obtained, and where, on moving days, additional teams can be readily hired.

When moving camp, breakfast was served an hour earlier than usual, the men in each tent then packed up their own things and got their own tent down. Certain men were then assigned to the office tent, and others to the cook tent, this latter with the cook's supplies being the last loaded and first unloaded and put up. A start was

usually made by 7 A. M. or a little after, and, with fairly decent roads, about 12 miles, the usual distance between camps, was covered, and the camp up by 2 P. M. The remainder of the day was spent by the party in checking estimates and in various office work, making stakes, getting firewood, etc. If necessary, some arrangement was generally made to keep the topographers and leveler working in the field on days when camp was moved, as they usually had some work to do in order to catch up.

Each locating party was organized as follows:

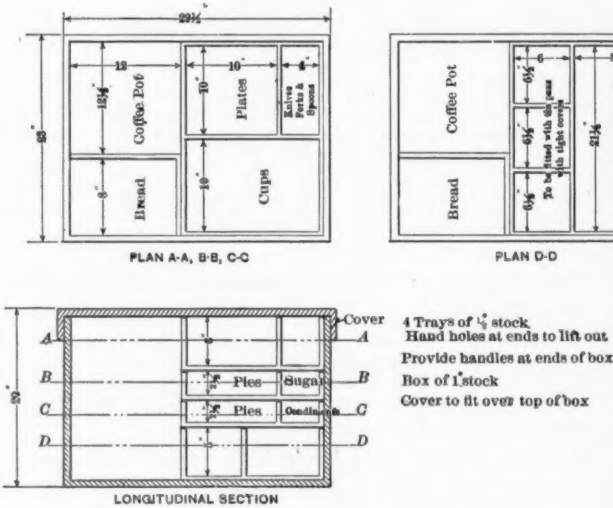
Locating Engineer	$150	to	$175
Assistant Locating Engineer	115	"	125
Transitman	90	"	100
Leveler	80	"	90
Draftsman	80	"	90
Topographers, two*	80	"	90
Rodman	50		
Head Chainman	50		
Rear Chainman	40		
Tapemen, two*	30		
Back Flagman	30		
Stake Marker	30		
Axemen (three to five as necessary)	25	to	30
Cook	50		
Cook's helper	20		
Double teams and driver, furnish their own feed, driver boarded in camp	65	to	90

Each man was supplied by the company with subsistence when in camp, but was required to provide himself with an army cot and sufficient bedding, and advised to provide a substantial canvas covering for the latter, an ordinary wagon cover, costing from $3 to $5, being the most easily obtainable and most satisfactory. The writer has always insisted, as far as possible, that men should equip themselves properly before starting out. The army cot takes up less space than any other cot, both when in use and when folded, and, if the bedding is properly protected, much cause for grumbling is removed on account of its becoming wet or dirty in moving camp, or on account of the tent leaking slightly, as the best will do at times.

* See note in regard to topographers, on page 126.

Each man's baggage, besides bedding, was limited to about what could be carried in an ordinary suit case; with the large party and equipment it was found that three good double teams had all they could do to move the outfit, and when the roads were bad it was sometimes found necessary to use an extra team.

Each wagon was required to be provided with a heavy canvas cover and at least one spring seat. The prices for teams varied with the locality and season of the year.



LUNCH BOX

FIG. 5.

The Locating Engineer was provided with a saddle-horse by the company, and an arrangement was made with the head teamster to feed and care for it.

Much of the success of an engineer in charge of location is due to his ability to handle readily the different characters that go to make up the party. Discipline, tempered with judgment, is of course essential, and a certain amount of formality is necessary. Seats were assigned at the table to each man in the order of his rank, and men were required to occupy their proper seats. Conversation

was in no way restricted, provided it was gentlemanly, except that no comments, either of praise or blame, upon the food on the table, were permitted. If any one had complaints they were required to make them to the writer out of hearing of the cook, and, on no account, were the men permitted in the cook tent except at meal times.

Frequent inspection of the living tents was made, and it was insisted that each man should make his bed and leave all his things in order before going to breakfast. The men assigned to the living tents were expected to divide between them the necessary chores, required to keep the tent in a clean and tidy condition. In sparsely settled country, considerable attention of this kind is necessary to insure the cleanliness and health of each man; the lazy habits of one should not be allowed to cause unnecessary discomfort to others.

Locating engineers reported directly to the Principal Assistant Engineer. A running account with each was opened with the head office of the railroad, cash being advanced from time to time on requisitions, properly O. K.'d. Provisions were bought by the Locating Engineer at the most convenient points, receipts being taken for all amounts paid out, and an expense account, on proper blanks, with the receipts attached, was sent in at the end of each month, the amounts of which, after being examined and O. K.'d by the Chief Engineer's office, being credited to the account at the head office.

It was found best, as far as possible, to establish a credit with one or perhaps two grocers, preferably wholesale dealers, near the proposed route, and deal with them exclusively, buying all provisions in case lots, or unbroken packages, where possible, thus getting wholesale rates.

It was expected that the men would be provided with good wholesome, plain food of the best quality obtainable. As a general rule, fresh meat and vegetables were difficult to obtain, and canned vegetables, dried fruit, and for meats, ham and bacon, had to be relied on to a great extent. While, of course, the final and principal object of the locating engineer is the location of a line of railroad, the fact should not be lost sight of, that good food and enough of it, properly cooked, is a very important factor in keeping up that *esprit de corps* which is absolutely essential to any degree of success in this particular work.

As stated above, the railroad company expected the men to be

properly provided for, but, at the same time, locating engineers were expected to see that there was no undue waste. The experience of the writer was that the expense for provisions for such a party as noted above should be between $250 and $300 per month, and any material increase over the latter amount should be questioned. In many instances, with his and other parties, it was found that a change of cooks often resulted in a large diminution in the expense account, with no difference in the quality or quantity of food provided. It was often found that the better cook was apt to be the more economical. Men, to whom problems of the commissariat were new, often bought supplies in too small quantities, thus paying as much as 10% more than was necessary.

The expenses of the parties, as noted in the tables of cost on page 134, show costs varying from $220 to $250 per month for provisions alone; the writer, however, has never made a conspicuous record for economy in the matter of buying provisions, opinions on what is proper and what is luxurious varying greatly; but, as with other things, the conditions of each case govern that case, and a happy medium should be striven for. Where the whole party is made up of well-educated and trained professional men, with the exception of the axemen, higher standards of living will be expected. Engineers on such surveys average, for week after week, 100 to 110 hours per week of actual work, as contrasted with 45 to 48 hours for the ordinary business man or office engineer, and the extra cost of providing them with whatever comforts can be reasonably obtained ought not to be objected to.

The expense accounts were carefully examined each month as sent in, and long experience and the many men in the field enabled the assistants at headquarters to determine very closely the reason for any particular account being above the average, and thus call the attention of the locating engineer to the cause in a definite manner.

The following is a list of groceries actually bought on starting a camp:

6 hams.	100 lb. flour, soft wheat.
6 pieces bacon.	100 " sugar.
50 lb. fresh beef.	5 " baking powder.
1 case eggs.	2 " tea.
25 lb. butter.	50 " coffee.
25 " lard.	50 " navy beans.
100 " flour, hard wheat.	25 " Lima beams.

12 lb. buckwheat flour.
5 " macaroni.
35 " cornmeal.
1 cheese (about 15 lb.).
12 packages oatmeal.
10 lb. rice.
100 cakes soap.
1 gal. molasses.
1 case condensed milk.
1 dozen tomato catsup.
½ " Worcestershire sauce.
1 gal. pickles.
½ dozen lemon extract.
½ dozen vanilla extract.
1 box dried prunes.
5 lb. raisins.
4 dozen assorted canned fruits.
1 case tomatoes.
1 " corn.
1 bushel potatoes.
1 kit salt mackerel.
20 lb. salt.
½ " mustard.
1 " pepper.
1 quart vinegar.
½ dozen yeast cakes.

Most of the lines to which the following methods most closely apply ran through a rather badly broken up, rolling country, with short cross-drainage, about three-quarters being wooded, and, in Indian Territory, very sparsely inhabited.

The low grades desired, 0.5 and 0.6%, compensated for curvature, and the nature of the country, involved considerable study of a rather wide range on either side of the proposed general direction of the line. No Government topographical maps of the country had been issued, and the only maps available were those published by the Public Lands Survey, showing fairly accurately, in relation to the section lines, the general location of the larger streams and rivers and some of the main roads.

On a proposed line of about 300 miles in length, a general route was established from previous reconnaissance with certain towns as governing points. Five locating parties were placed in the field, each assigned to about 60 miles. The results desired to be obtained on the location were:

First.—To establish the fact that a practical line could be obtained with ruling grades of 0.6%, or if not, what was the lowest practical ruling grade that could be obtained;

Second.—To be sure that the line obtained was such that no other line could be built through the same country with the same or better ruling grades, with less expenditure, at the same unit prices;

Third.—To keep close control of the work and results of all the parties from a central headquarters;

Fourth.—To have, on the completion of the survey, complete right-of-way maps, estimates of quantities and cost, profiles showing in detail the exact nature of the work, so that contractors could bid intelligently, and work be started at once if necessary;

Fifth.—To keep the cost of the surveys as low as possible consistent with obtaining the above results.

As stated, it was first desired to establish the fact that a reasonable line could be obtained with maximum grades of 0.6% throughout the entire route. Each locating engineer, therefore, was instructed to get one line through as soon as possible on which this condition was established, simply noting carefully, but leaving for future investigation, other and perhaps better lines or deviations from the line first run.

It is advisable, where possible, for the Locating Engineer to take a trip over the proposed line before the party gets into the field, but this cannot always be done, as in the particular case referred to. A reconnaissance by the Principal Assistant Engineer, however, had indicated the location of the first camping place and the first work to be done. The party, therefore, was immediately set to work under the direction of the Assistant Locating Engineer, while the Locating Engineer made a more careful reconnaissance over that part of the line assigned him, making the round trip of approximately 120 miles in 4 days.

The trip was made in an open spring wagon; observations with hand-level and compass were taken, and a full sketch was made of the road, showing all branches from it, stream crossings, houses, of which there were few, sketched topography and all the local names that could be learned. Distances were estimated very closely by observing the time of passing all points which could be identified, such as section corners, houses, fences, streams, etc. Fig. 6 shows a page of the notebook used, and Fig. 7 a portion of the completed map.

On this trip, at the end of the line, which was at a town of moderate size, arrangements were made with a local grocer and provision dealer, so that, when it became more convenient to send to that end for supplies than to the town nearest the starting point, all that would be necessary would be to send the teamster with the necessary order.

In working out the details of the location, nearly the whole time of the Locating Engineer was spent looking up the general broad features of the country, the actual work of looking after the field party and running the preliminary line devolving practically entirely on the Assistant Locating Engineer.

In this connection the writer would call attention to a recent article

on "Railroad Reconnaissance" in *The Railway and Engineering Review,** by Willard Beahan, M. Am. Soc. C. E., which describes quite fully and well the work required of the engineer in charge of location, both on the preliminary reconnaissance and in the conduct of the work in the field.

TYPICAL PAGE OF RECONNAISSANCE NOTE BOOK.

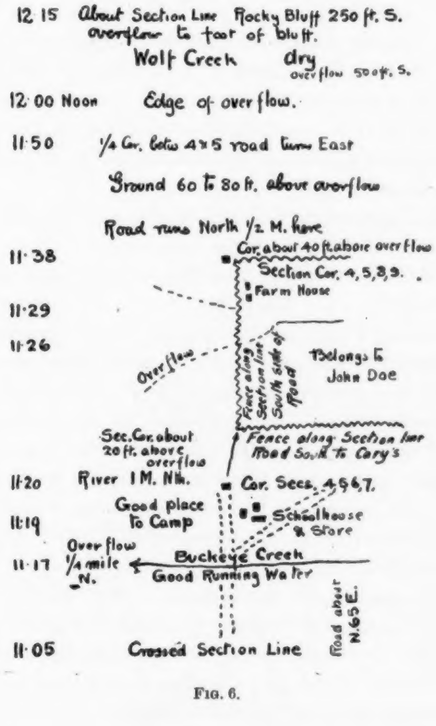


FIG. 6.

In laying out the work for the Assistant, considerable use was made of rough sketches showing the general topography of the country, and, each evening, with the aid of these and the map and profiles of the lines already run, the following day's work was ar-

* March 12th, 1904.

ranged. During the day, the Locating Engineer found the party at more or less frequent intervals, saw that the line was being run as desired, and made such modifications or revisions of instructions as might be necessary.

It is deemed by the writer most important that the Locating Engineer should be absolutely free at all times, either to stay in camp and keep the office work up to date, to make reconnaissances for any distance ahead, to be with the party at critical points, in fact, to be wholly in an executive position and not tied down to any details. A conscientious study of the country will keep a good man very fully occupied with this work, with "office hours" from 6 A. M. to 9 P. M., and, by working overtime occasionally and planning the work carefully, he will be able to get a couple of hours sometimes on Sundays to write home to his friends.

In the field, the Assistant Locating Engineer had general charge of the party, and, on preliminary work, was responsible for the proper working out of the details of the line from the general location determined on by the Locating Engineer. A special point was made of actually running a preliminary line, getting the topography on it and making a projected location and profile wherever there was a reasonable possibility of a good line existing. Especially is this important in wooded country, where the difficulties of comparatively small details are apt to be magnified, and a good location missed because of the actual physical difficulties of getting over the ground, or of sizing up the topography as a whole. In this connection, it is believed by the writer that little time should be wasted in the field in getting the preliminary so close to where the final location may come that it will show up a good profile. The point of prime importance is to get the preliminary close enough so that the projected location will fall well within the limits of the topography. Of course, the difference between good and bad judgment will show here, as everywhere, but the point is, not to have the party sitting around waiting, and going back to refine the small details on the preliminary; all this can be done to much better advantage on the map, where a broad general view of the whole line can be taken.

On making the final location, the Assistant Locating Engineer obtained from the map the necessary data to connect the located line with the preliminary, and to lay it out on the ground as projected,

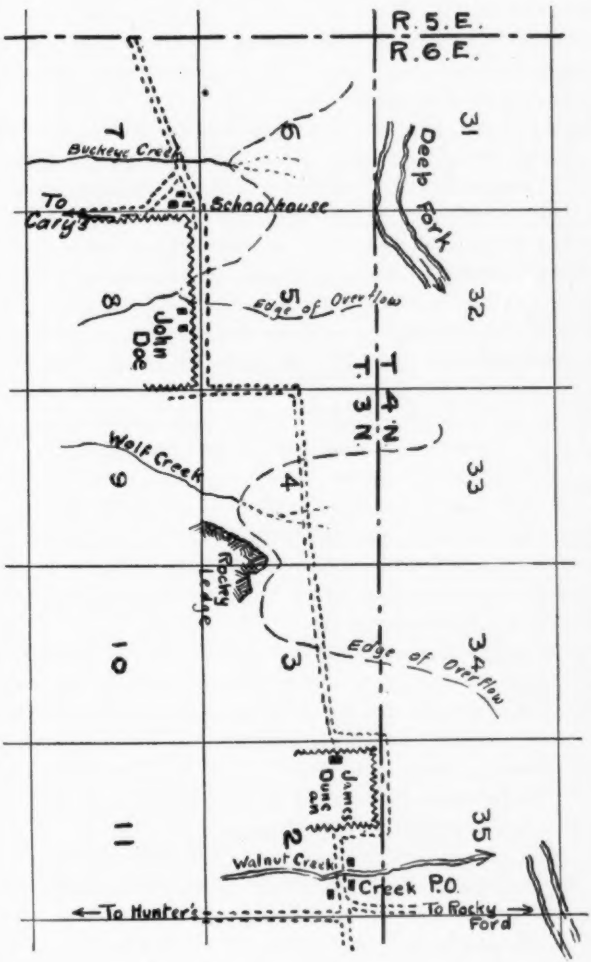


Fig. 7.

Finished Map as Drawn from Notes as Shown in Fig. 6.

which data he kept in a notebook used for that purpose only. He had with him in the field a copy of that portion of the projected profile covering the work in hand. The points on the located line were fixed by horizontal distances from the hubs on the preliminary, and also, and especially where the slopes were at all steep, by vertical distances, and points of elevation fixed by the leveler.

It should be noted, especially, that, in anything like rough country, the essential point, in running in a location from a line projected on a map, is to reproduce the projected profile rather than the projected line. After deciding that the projected profile is the one to be obtained, the location must be passed through all the controlling points whose vertical distance (or, rather, elevation) is such as called for by the projected profile and determined by leveling from some known elevation (a hand-level will generally give close-enough results).

The actual profile obtained by the leveler was platted in the field at critical points and compared with the projected profile to see that the proper results were being obtained. Absolutely correct topography is not required and is not essential, if the vertical method is adhered to in laying down the location. Many engineers have objected to topographical work on a railroad location on account of the expense necessary for accurate topography, but this latter, and the expense incident to it, is entirely unnecessary if the locating engineer in charge of the party knows how to place his projection on the ground, so as to equalize any slight errors made.

The difference between theory and experience, here, is that theory would require an absolutely accurate preliminary and topography, while experience shows that neither is essential, if proper methods are adopted in getting the line on the ground. The writer believes that many locating engineers lose sight of the fact that the location should bear a certain relation of position to the local topography, rather than a relation to a certain geographical position on the map.

It was considered advisable to avoid equalization stations, on the final location, as the result of revisions of the line, as much as possible, and every care was taken to get the line right in the first place. As the preliminary lines form the basis on which, not only the projection of the location depends, but also of the topographical map, which forms the basis of the final completed record of the survey, it is essential that, without wasting time and money on unnecessary refinements of accuracy, all due precautions should be taken to prevent unnecessary errors.

PRELIMINARY LINES. TRANSIT NOTES.

Line "S"

Sta.	Dist.	Defl.	Course	Calculated Course
⊙ 61+40				
⊙ 58				
	1020		S.53.00 E.	S.53.56 E.
△ 51+20		2.00 L.		
	402		S.52.00 E.	S.51.56 E.
△ 47+18		5.30 R.		
	368		S.57.00 E.	S.57.26 E.
△ 43+50		4.00 R.		
⊙ 36+20				
⊙ 29				
	1950		S.61.00 E.	S.61.26 E.
△ 24		3.00 L.		
	400		S.58.00 E.	S.58.26 E.
△ 20		7.00 L.		
	2000		S.51.30 E.	S.51.26 E.
⊙ 12+31				
	6140			
=0 "S"		21.15 R.		
△ 446+90 R.				
	970		S.72.30 E.	S.72.41 E.
△ 437+20 "R" Backsight				

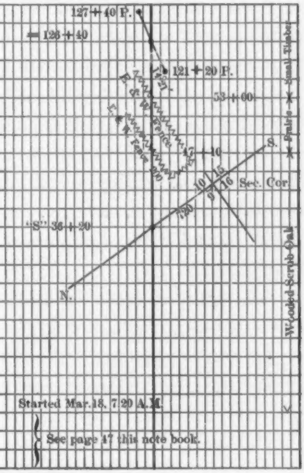


FIG. 8.

FINAL LOCATION. TRANSIT NOTES.

Sta.	Dist.	Defl.	Course	Cal. C.
△ 755+19.21	P.C.3° L.			
⊙ 751+38				
⊙ 748+25				
⊙ 743+20				
⊙ 742				
	1681.27		N.42.30 E.	N.42.46 E.
△ 738+37.94	P.T.	5°14′00″		
738		4.51.14	∠	10°28′
737		3.51.14	Rad.	2864.93
△ 736		2.51.14	S.T.	262.40
735	523.33	1.51.14	L.C.	523.33
734		0.51.14		
△ 733+14.61	P.C.2° R.			
⊙ 729+50				
⊙ 723				
	1276.40		N.32.10 E.	N.32.16 E.
⊙ 720+38.21	P.T.			

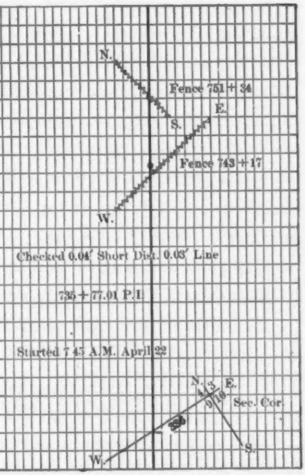


FIG. 9.

All angles were checked by doubling and by compass readings, the rear flagman kept a list of all hubs to check the transitman, and the rodman worked out in a separate book all elevations of turning points, thus checking the leveler, this latter being most important.

In settled cultivated country, on both preliminary and location lines, hubs were set at all fences, as both stakes and hubs in the fields are frequently, and, in fact, usually, pulled up or destroyed soon after the line is run, while the hubs at the fences remain, enabling the ready re-establishment of the line. All tangents were run to an intersection, the P. C. and P. T. of all curves being set before the curves were run in.

Surveys were made of, and levels run on, all railroads crossed, for one mile on either side of the line.

An exact copy of the transit notes was made every night in an office notebook. Figs. 8 and 9 show typical pages of transit notes on preliminary and final location.

The head chainman is responsible to a greater degree perhaps than any other one man in the party for the progress of the work, that is to say, the actual physical progress, after the direction of the line has been determined. He must be a hustler from start to finish and for 10 or 11 hours per day, and, at the same time, be accurate; it is no use being careful about the instrumental work and the platting, if the chaining is not well done, and nothing is more annoying than to plat side lines forming traverses and find that they will not close, or to run in curves on location and not have them check. Chainmen, as a rule, are not college graduates, but are picked up in the country where the survey is carried on, and probably have had some experience, and there is no point where the engineer in charge of surveys can use his time to better advantage at the beginning of the work than in getting the chainman to do good work, not necessarily measuring to thousandths, but accurately enough for the work to be done.

There were three axemen (except in one or two cases of very heavy timber for several miles, where two extra men were put on for a short time); in wooded country, all chopped, but in that more or less open, one man helped the stake-marker by driving the stakes.

The writer has found it advisable to pick out one axeman and place him in charge of the others, spending time enough with him at the beginning of the survey to break him in thoroughly to the idea of

keeping on line and cutting only what is necessary to get the line through; the right kind of a man will soon learn what is required, and save a great deal of time.

Leveling.—Bench-marks were established every mile on preliminary lines and every half-mile on final location, care being taken to have these latter convenient for construction purposes, especially near bridge openings, etc., and away from all danger of being interfered with by the excavation. The elevation of each bench-mark was plainly marked on or near it.

TYPICAL PAGE. LEVEL NOTES.

				Line "S"		
Sta.	B.S.	F.S.	H.I.	Rod	Elev.	March 22. Started
					749.20	B.M. 30 ft. L. of 439+40 R. on White Oak
	10.24		759.44			N. Bk. 14 page 39
S. 0				6.2	53.2	
1				8.3	51.1	
+35				10.4	49.0	
+40				10.8	48.6	
2				8.4	51.0	
3				4.1	55.3	
T.P.		1,06			755.38	On large boulder 3 R. of 3+45
	7.48		765.86			
4				5.4	60.5	
+65				1.1	64.8	
5				3.2	62.7	
6				5.1	60.8	
+30				7.8	58.1	
+60				10.1	55.8	
7				9.3	56.6	
8				8.1	57.8	
9				6.2	59.7	
T.P.		6.23			759.63	Peg at Sta. 9
	9.54		769.17			769.17
10				6.4	62.8	749.20 19.97
	27.26 7.29 19.97	7.29				

FIG. 10.

Level notes were kept in the form shown in Fig. 10.

On the final location at all bridge openings ravine sections were taken, all plusses being measured in with a cloth tape, and a very

NOTE.—In using the metric system, stakes are driven every 20 m., and numbered from the beginning, 0, 2, 4, 6, 8, 10, etc. Curves are run in, the same as when using 100-ft. stations, except that the deflections are for 20 m. instead of 100 ft.; as 20 m. equals very closely $\frac{2}{3}$ of 100 ft., the radius of an 8° curve metric equals approximately a 12° using 100-ft. chords. Tables of radii of metric curves are given in Henck and elsewhere.

Levels are taken as usual, ordinary readings to centimeters and target readings to millimeters. Profiles are platted on metric profile paper, the smallest divisions being millimeters, giving a profile, at the scale ordinarily used, about the same as that platted on Plate B paper.

Maps are usually platted on a scale of 1: 5 000, about equal to 400 ft. to 1 in.

careful profile was obtained which was afterward platted on a scale of 10 ft. to 1 in. (see Fig. 11).

Topography.—This was taken for 300 ft. on each side of the line, in cross-section books, ruled, on a scale of 8 ft. to 1 in., in 100-ft. squares, subdivided into 10-ft. squares.

In the field a hand-level and a light wooden rod, 2 in. by ¾ in. by 12 ft., marked every ½ ft., were used, and distances out were paced, or measured with a cloth tape, according to the nature of the ground, and 5-ft. contours were located and sketched in, care being taken at angle points to get sufficient information to connect the contours properly on the map.

As previously noted, absolutely correct topography was not required, and distances were not taped except in cases of very steep slopes; the accuracy with which distances up to 500 ft. can be paced, by any one accustomed to doing it, and levels carried the same distance by hand-leveling, was well brought out in an account of the survey of the Biltmore Estate, by J. L. Howard, M. Am. Soc. C. E.,* and the writer's experience confirms that of Mr. Howard.

Each day the books used on the previous day were left in camp, the work was platted by the draftsman, and other books were taken out. Each book was indexed each night, and each day's work dated at beginning and end.

On the final location, a sounding party was organized, with Topographer No. 1 in charge, and two or three laborers; soundings were taken in all the cuts and at bridge openings, ship augers and steel drills being used most generally. The former were welded to a 12-ft. steel rod with an adjustable handle. These soundings, taken with augers, determined very closely the character of the material in the cuts. In country where boulders might be likely to be encountered to any great extent, either in a clay or gravel formation, this method would not answer; but through the country traversed, as proved on construction, the estimates made from the results of these borings and a close study of the country were quite near the final estimate.

In one or two instances, of important structures and very deep cuts, a regular well-drilling outfit was secured, and the work looked after by a man engaged for the purpose under the direction of the Locating Engineer.

* *Journal of the Association of Engineering Societies*, Vol. XVIII.

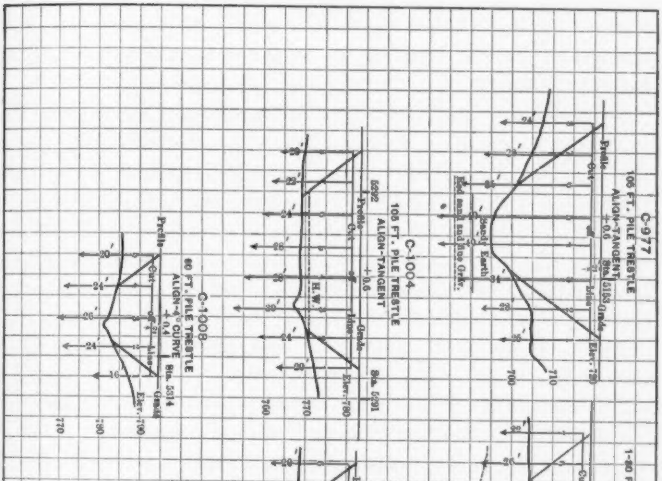

C-977
105 FT. PILE TRESTLE
ALIGN-TANGENT
C-972 TUSKYOKA CR.
1-80 FT. DECK GIRDER-1-PIER-1-ABUT. AND 105 FT. WEST PILE APP.
ALIGN-TANGENT
C-1004
105 FT. PILE TRESTLE
ALIGN-TANGENT
C-996
75 FT. PILE TRESTLE
ALIGN-TANGENT
C-992
60 FT. PILE TRESTLE
ALIGN-4° CURVE
C-1008
60 FT. PILE TRESTLE
ALIGN-4° CURVE
CHOCTAW OKLAHOMA & GULF R.R.
RAVINE SECTIONS
ON
CHICKASAW EXTENSION

FIG. 11.

The second topographer, supplied with a transit and assisted by the two tapemen, determined the drainage areas, located the property lines and section corners, got names of property owners, etc. This method was found much more economical than to have the whole transit party held up while the transitman and chainmen were getting this information.

With the information thus obtained by the two topographers, the profiles and map of the final location, which were finished within a few days of the completion of the survey, contained all the information necessary to proceed with the construction.

It should be noted here that there were exceptional circumstances in connection with this survey which made it desirable to employ two topographers. Ordinarily, one is sufficient, and a good man will easily take 80% of the topography. Generally, about moving day, the topographer is a day or two behind, in which case the whole party is broken up into topographical parties, and the work cleaned up to the end of the line in a part of a day. Also, when only one topographer is available, when the final location is run in, the Assistant Locating Engineer is occupied about two-thirds of the time in getting land lines, drainage areas, etc., and assisting with the office work, while the Locating Engineer looks after the actual running in of the line. This is ordinarily the most economical arrangement, but, in the survey referred to, it was necessary to rush the work, regardless of the slight extra expense of using men occasionally at a disadvantage.

In carrying out the third requirement, of keeping headquarters in touch with the work, a weekly report was made by the Locating Engineer; and the following maps, etc., were kept in shape and up to date:

On preliminary lines: General map, scale 5 000 ft. to 1 in., at the bottom of which was a condensed profile of the projected location, scales 1 000 and 100; detail map, scale, 400 ft. to 1 in.; profiles of preliminary lines and profile of projected location, Plate A paper, scales 400 and 20; profile of projected location on tracing profile paper.

On final location: Line inked in on 400-ft. map, and drainage areas shown; right-of-way map, scale 2 000 ft. to 1 in. (required only in Indian Territory); maps of station grounds, scale 100 ft. to 1 in.; final profile on Plate A paper; final profile on tracing profile paper, in 10-mile sections; ravine sections of all bridge sites.

Office Work.—The first duty of the field draftsman was the preparation of the general map on the 5 000-ft.-to-1-in. scale, from the best available sources, covering the whole of the country in which the proposed line might lie. In most of the country in the West, where topographical maps have not yet been prepared, the Government maps of the Public Lands Surveys, showing the section, township and county lines, town sites, and the location of the main drainage, will form the basis of this map.

This 5 000-ft. map and profile are absolutely essential to a broad comprehensive study of the line as a whole; it can be readily seen from this whether or not a good general direction is being maintained, and the general relation of the line to the surrounding country is shown. Such a map, with the omission of the preliminary lines, is of considerable aid to contractors in computing the haul of construction material and for other uses; it is also generally sufficient to accompany such reports as are made to the higher officials, in fact, it gives them a more comprehensive idea of the line than a more detailed map. Plate XII shows a portion of a 5 000-ft. map, but shows the located line only; the writer regrets that he has not available a map as described, showing also the preliminary lines and condensed profile.

A tracing was made of this map and, as soon as completed, sent to headquarters; from day to day, the preliminary lines run were platted on it, and, also, the projected location and profile, as they were made.

At the end of each week a tracing of the portion of the map showing the additions made to it during the previous week was sent to headquarters, where the information was transferred to the original tracing.

The weekly report which accompanied this map explained in quite full detail such points in connection with it and the work as seemed to require explanation. It explained, in particular, the natural features of the country, the availability or otherwise of timber (especially for ties), stone, sand, etc., the condition of roads, water supply, and, in general, the work of the party during the preceding week.

All the preliminary lines run during the day were platted on the 400-ft.-to-1-in. map in the evening, from the calculated courses and distances; no platting from deflection angles was allowed. The work of the draftsman was checked by the Assistant Locating Engineer.

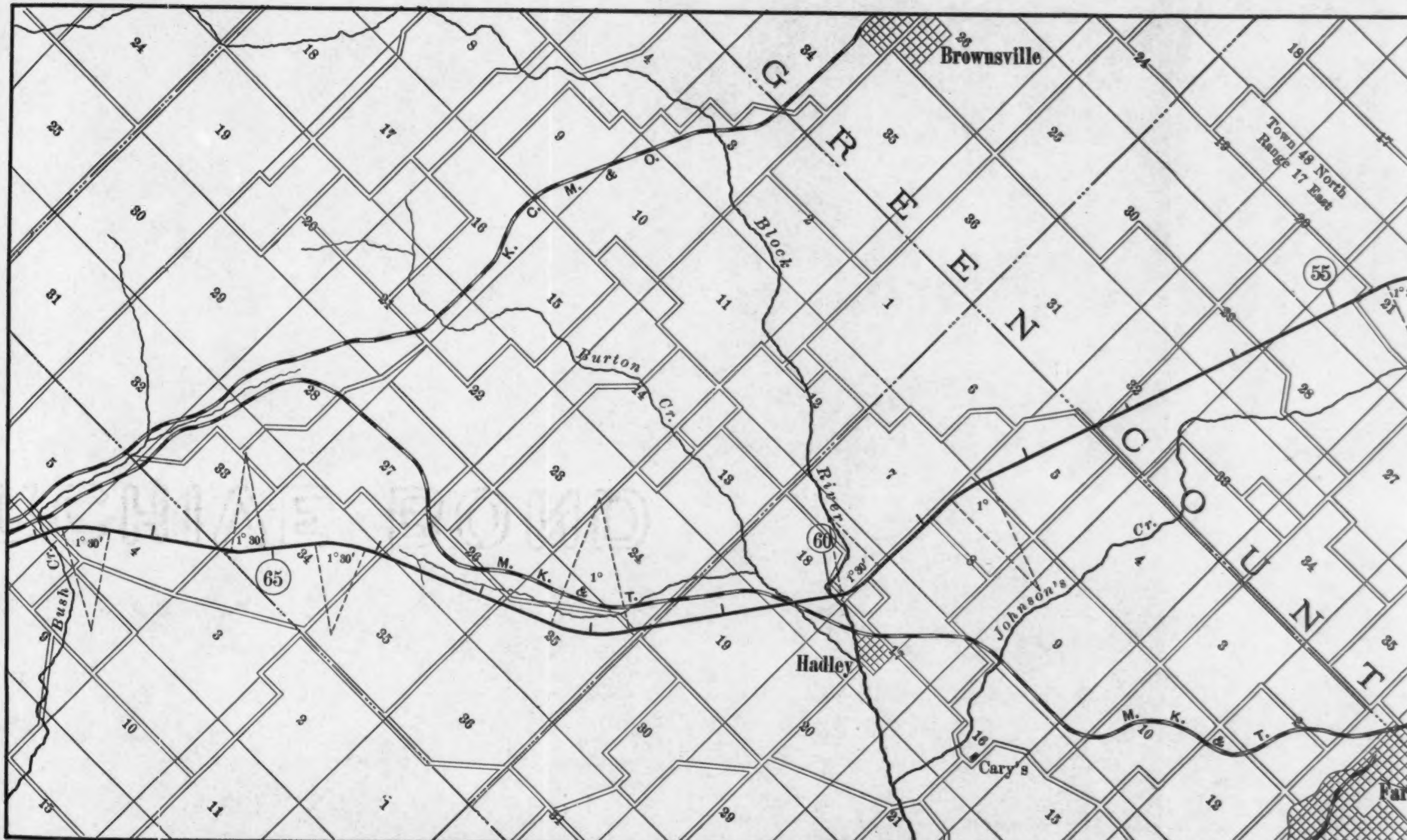
The first thing the following day, the line was inked in, in red, station numbers were marked at all angle points, the topography taken the previous day was platted and inked in immediately in black. Black was found preferable for the contours, as it stood the erasing of the projected lines better than colors. Valleys were indicated by a light blue line drawn through the lowest points, thus making the topography stand out and easier to read. It was found necessary at times to have one topographer stay in half a day and assist the draftsman by platting the topography, in order to keep all the parts of the work co-ordinated.

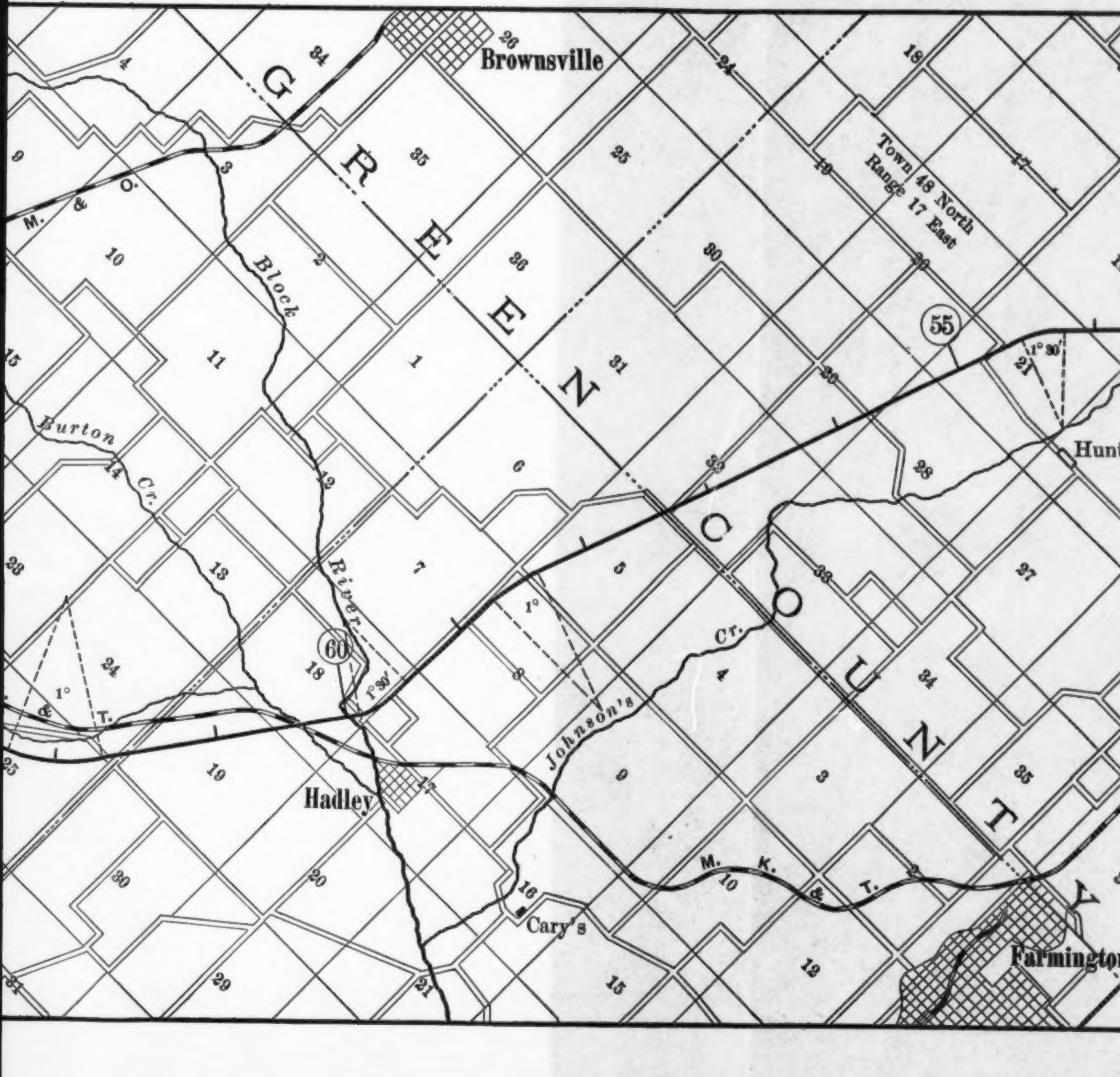
With the large party used, more than 4 miles per day of preliminary were averaged, and it was absolutely necessary to keep the topography close up, in order that the projected location should not get behind. As a rule, as the map was being used in the evening and the early part of the day, the Locating Engineer endeavored to get back to camp about 4 P. M. and get the projected location up to date before supper. Separate sheets, as advocated by Mr. Wellington in his "Economic Theory," may have some advantage in this respect, but the writer prefers the map on a roll of 36-in. paper. There are few places in difficult country where it is not necessary to run more than one line, and it has always seemed to the writer to detract from a general, broad, comprehensive study of the lines, as a whole, to have them scattered around on separate small sheets of paper.

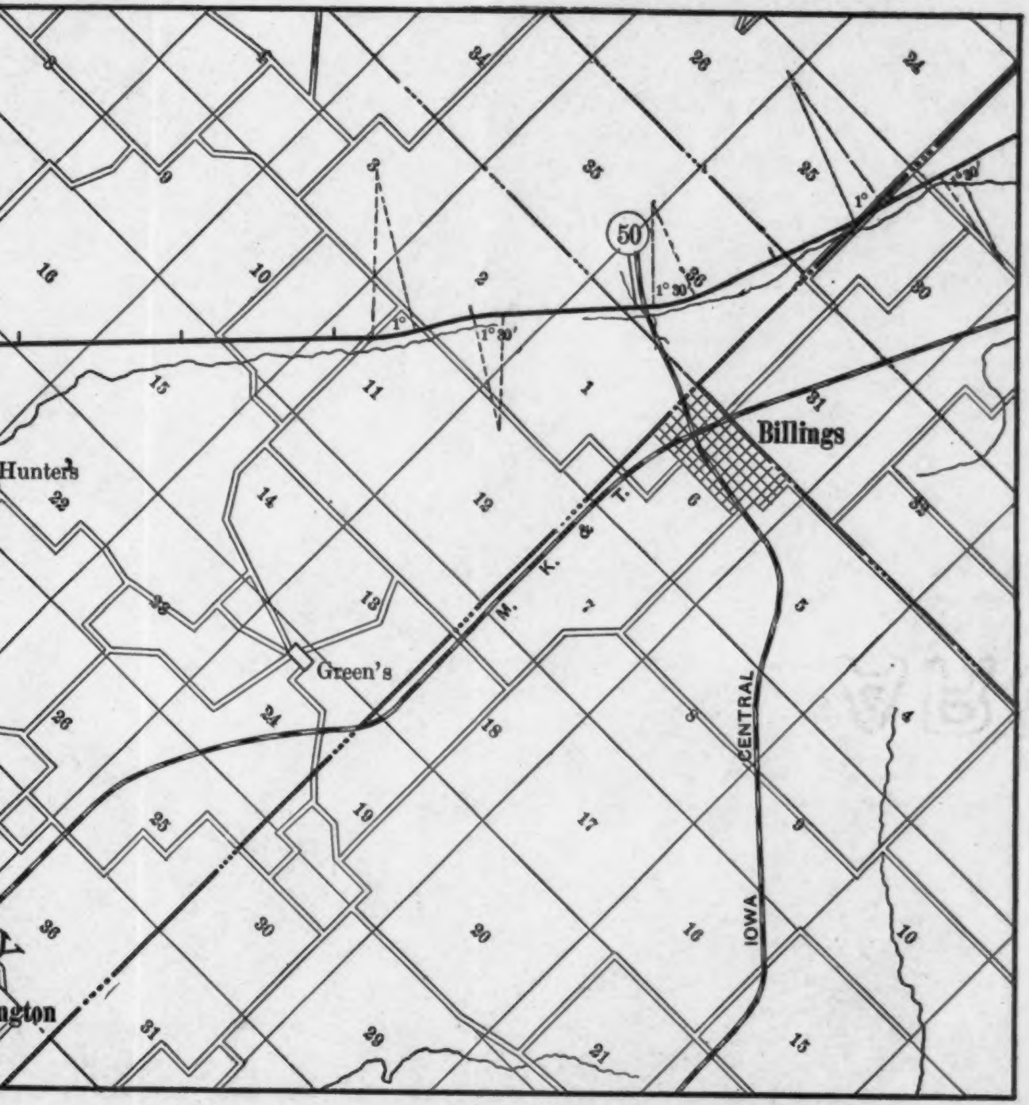
Necessarily, also, by the separate-sheet method, the lines and topography must start right from the edge on one side of the sheet, where it is very likely to be torn; on the 36-in. rolls used on this survey, no topogrephy was allowed within 6 in. of the edge of the paper, except possibly at a point where it just came near the edge and immediately receded from it.

It was absolutely required that the projected location should fall within the limits of the topography, that is, within 300 ft. of the preliminary line, and if, for any considerable distance, it was more than 200 ft. from the preliminary line, it was often deemed advisable to cover this with another line.

In making the projected location, a sheet of tracing cloth, on which was drawn in ink to the scale of the map the curves proposed to be used, with tangents at the ends, was found very useful in fitting the alignment; 100-ft. stations were marked on these curves on the tracing


Brownsville
G R E E N C O U N T
Town 48 North
Range 17 East
K. C. M. & O.
M. K. & T.
Block River
Burton Cr.
Bush Cr.
Johnson's Cr.
Hadley
Cary's
55
60
65


Brownsville
GREEN COUNTY
Town 48 North
Range 17 East
M. & O.
Block River
Burton Cr.
Johnson's Cr.
55
60
Hadley
Cary's
M. K. & T.
Farmington


Billings
Hunters
Green's
M. K. & T.
IOWA CENTRAL
50
1°
1° 30'
ngton

cloth, so that, when laid above the topography sheet, pieces of profile could be readily read off. No P. C. or P. T. was allowed to come within 400 ft. of the end of a bridge; all curves were of even degree of curvature, being either 1, 2, 3 or 4°, and all grades were in even tenths of 1 per cent.

The writer objected to the limitations of the degrees of curve at first, believing that a nicer adjustment could be made by using any degree, with indices of odd minutes when necessary, that seemed at first to fit the ground better, but by being compelled to use the even degrees, found afterward that this could be done almost invariably just as well, but required perhaps more study of the situation. Of course, no rules of this kind can be absolutely iron-clad, and the Principal Assistant Engineer at times modified them himself, but it was considered advisable to make them binding as far as the locating engineers were concerned.

The projected location being made and penciled in on the 400-ft. map, the profile was taken off and the grade line fixed, grades being kept in even tenths o 1%, except where compensated for curvature, and even then, if possible. Of course, in long stretches of ruling grade where often every inch counts, hundredths of 1% rates occurred where compensation was made. Breaks for compensation were made at the even stations nearest the ends of the curves.

All bridges and culverts were located on this profile, the probable quantity of classified excavation in each cut was indicated, and an estimate made of each mile. The classification of the material in the cuts, as shown on this projected profile, was made by the Locating Engineer from his observations of the surface indications; of course, this was only approximate, but was quite close.

Specifications and standard plans of all structures, with tables of constant quantities, were furnished by the railroad, and the excavation and embankment quantities were figured from tables of level cuttings for the standard roadbed sections used. A useful device for scaling the quantities from the profile was made by taking a piece of the same profile paper used for the profile and marking, along the edge at each foot, the quantities corresponding to the height, starting at 0 (see Fig. 12).

As each 10 miles of this profile of the projected location was completed, a tracing on tracing profile paper was made, showing the

estimated quantities of each mile and, also, on a regular estimate blank, a summary of quantities, and a summary showing:

Total length,
" degrees of curve,
" length of tangent,
" percentage of line on curve,
Maximum curve,
" grade,
Total rise (in the direction of the line),
" fall (in the direction of the line),
" length of bridging,
" cost,
Average cost per mile,
" number of cubic yards per mile.

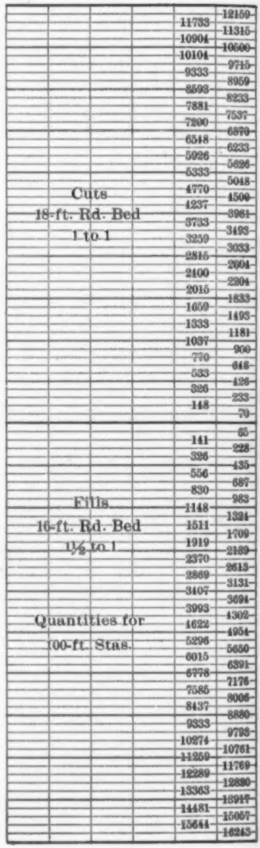
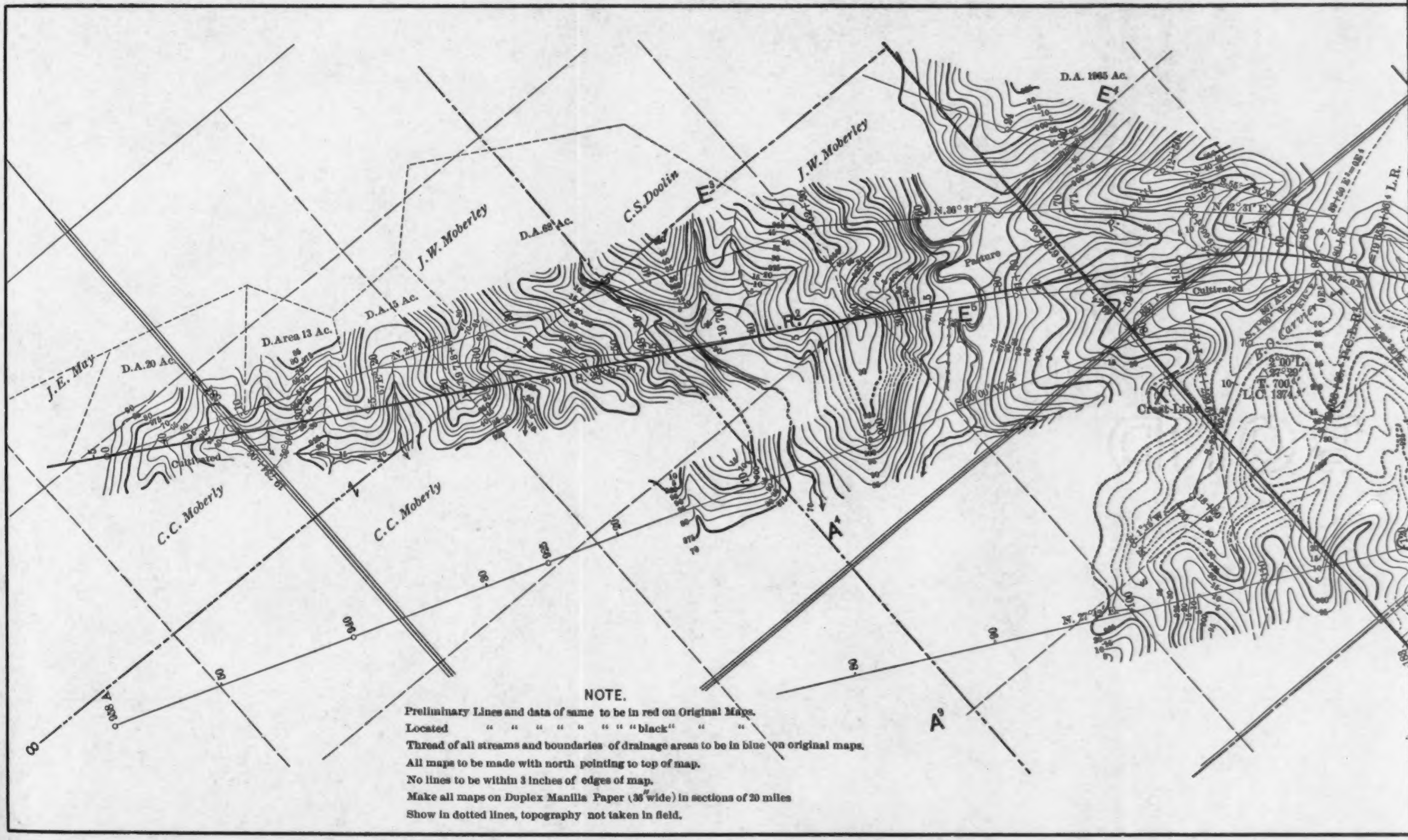


FIG. 12.

These tracings of 10 miles each, with the estimates and summary, were forwarded to headquarters as fast as completed, and, on the completion of the line, an estimate and statement similar to the above, covering the whole line, was sent in. The original profile was made in 25 to 30-mile sections.

From time to time the Principal Assistant Engineer visited each locating party in the field, and thus kept in touch with the work and results of each. At the same time, the 5000-ft. map and profile, etc., kept the record at headquarters complete. All projections adopted for location were examined and approved by the Principal Assistant Engineer, and, after approval, no deviation was permitted without authorization. By this means a detailed study of the line was possible; much more so than when viewed only on the ground by those superior to the Locating Engineer.



NOTE.

Preliminary Lines and data of same to be in red on Original Maps.

Located " " " " " " " " "black" " "

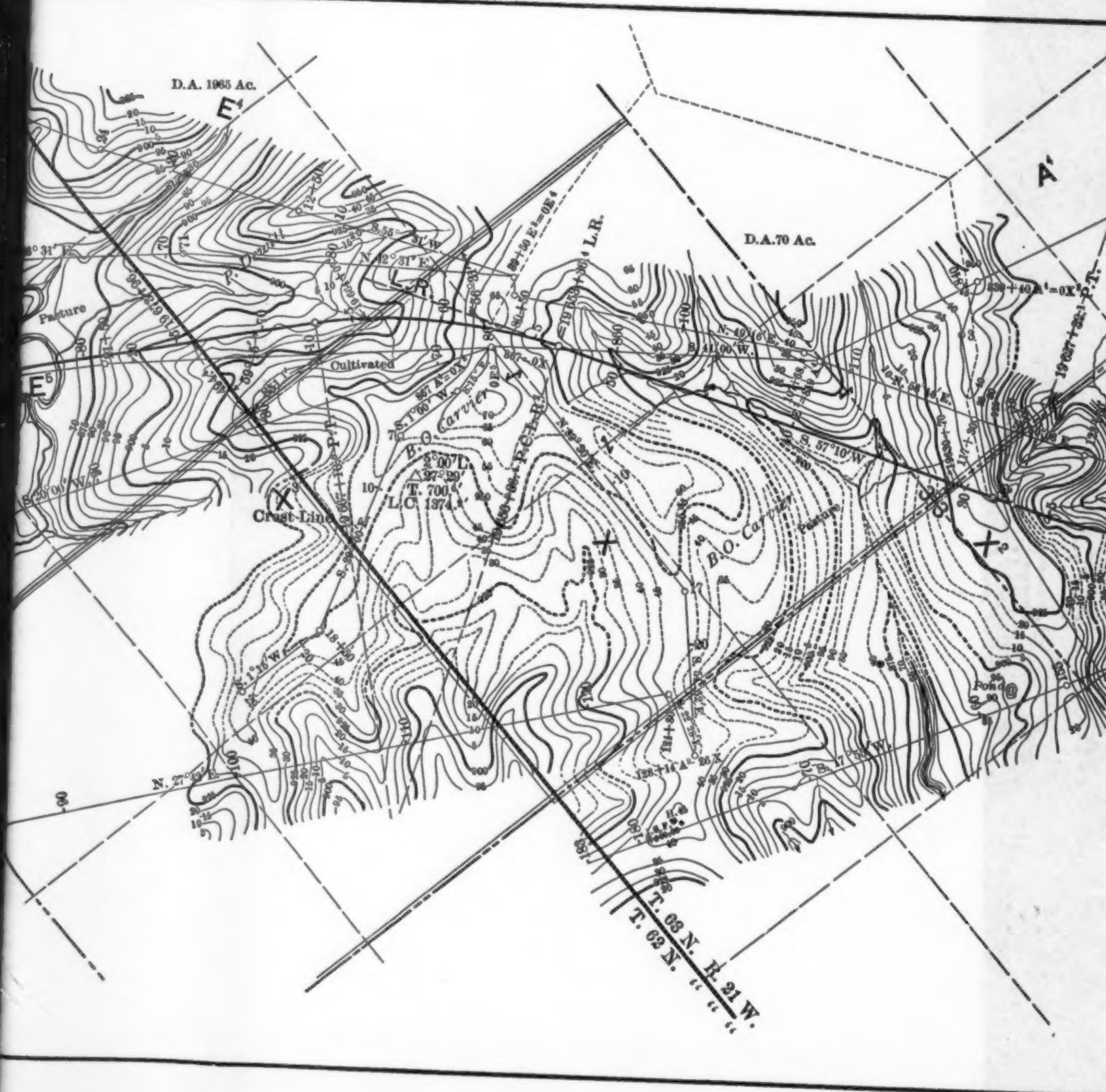
Thread of all streams and boundaries of drainage areas to be in blue on original maps.

All maps to be made with north pointing to top of map.

No lines to be within 3 inches of edges of map.

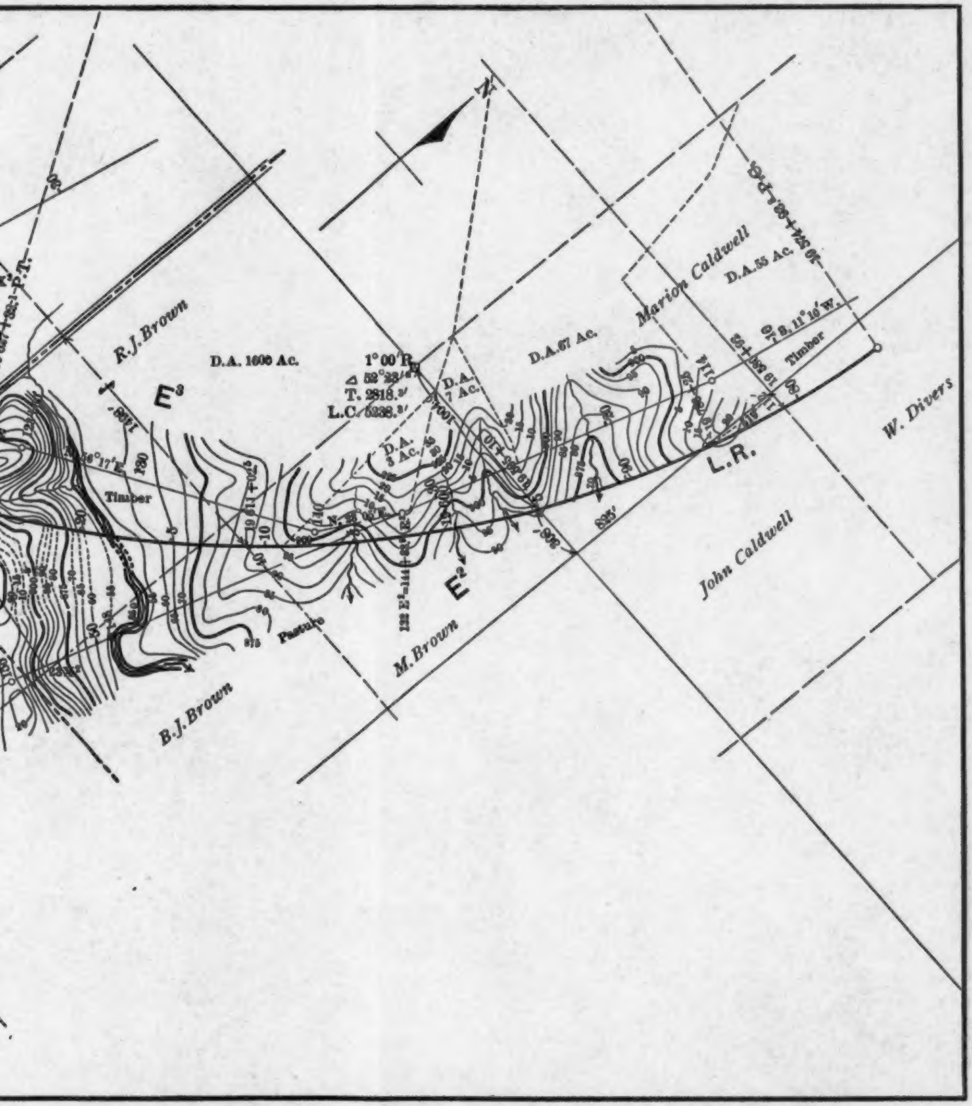
Make all maps on Duplex Manilla Paper (36" wide) in sections of 20 miles

Show in dotted lines, topography not taken in field.



D.A. 1965 Ac.
D.A.70 Ac.
Pasture
Cultivated
Crest-Line
Pond
L.R.
B. O. Carrier
T. 63 N. R. 21 W.
T. 62 N. " " "

PLATE XIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 991.
LAVIS ON
METHODS OF RAILROAD LOCATION.



On the particular line in question, it was decided, by the time the different preliminary lines were nearly connected, that a 0.5% grade was possible for the whole length of the line, and instructions were given to make this the ruling grade on the final location.

Instructions were given the locating engineers to spend all the time necessary on investigations, to be sure they had the best line through the country traversed before putting in the location, on the ground. The writer recalls one stretch of line, about 16 miles in length, where he spent nearly three weeks, running more than 80 miles of preliminaries, besides the original preliminary and projected location, before the final line was decided on. A second projected location saved a mile of distance over the first, besides eliminating much curvature and rise and fall, and, but for the very positive instructions received to exhaust every possibility, and the receipt, about this time, of a letter from the Principal Assistant Engineer, who knew the difficult nature of the country, reiterating his caution, this line would have been run in. Other lines were run, the final location effecting a saving of more than $30 000 in estimated cost of construction, and eliminating many degrees of curvature and more rise and fall.

It seems hardly possible, in view of this, which is only one case out of thousands, that anyone contemplating the construction of a railroad should hesitate to spend sufficient money on surveys, but all engineers of any extended experience know how difficult it often is to get either sufficient time or money to do this work thoroughly; and, as a result, how very much more the cost of the needless construction is likely to be than that of the surveys. Still, the writer believes it is often the fault of engineers in charge of work that this is so. Men now-a-days investing their money in any project of merit are as a rule level-headed business men who would be willing to furnish all the money necessary for proper surveys, if the matter were presented to them in the proper light.

As the final located line was run in, it was inked in on the 400-ft. map, radii of curves were drawn, stations of P. C. and P. T. marked, and calculated courses of tangents from observations of Polaris, length of tangents, the degree of curve, central angle, and the length of semi-tangents noted at each curve; drainage areas, as definitely determined by the topographer, were dotted in, and areas noted; as were also property lines and owners' names, thus making the map a complete record (see Plate XIII).

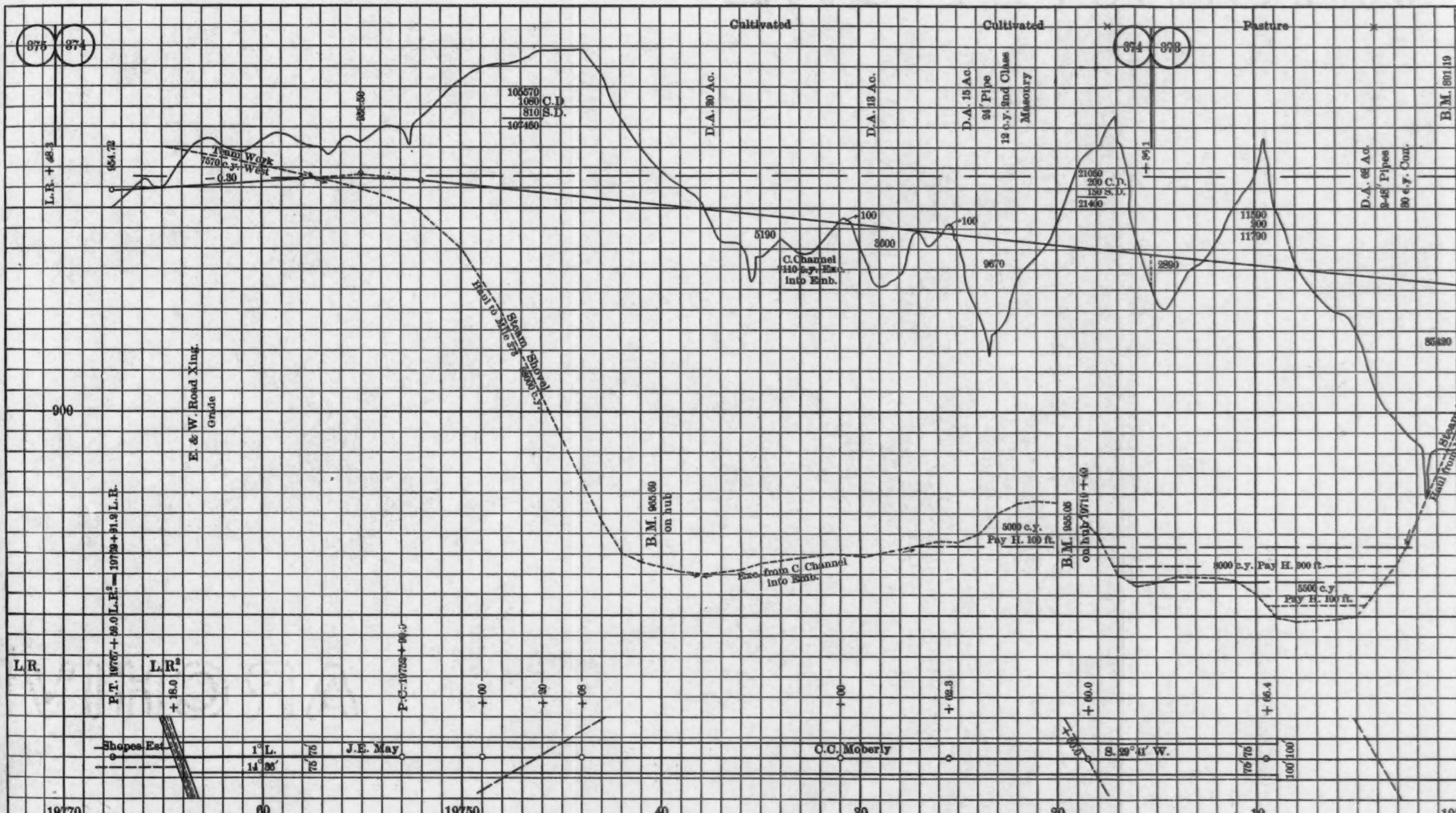
In Indian Territory a right-of-way map, on a scale of 2 000 ft. to 1 in., was made, showing the alignment, station of P. C. and P. T. of curves, central angle, degree and length; also length and calculated course of tangents, all property lines and plusses to same, and property owners' names, where land had been allotted; ties to all section or quarter-section corners nearest the line, and notes in pencil where extra width for large cuts or fills might be necessary, the final width desired being added at headquarters. This map was required in Indian Territory only, to meet the Government requirements, the right of way being obtained by filing such a map with the Secretary of the Interior.

As the final profile and the ravine sections were platted, they were taken into the field by the Locating Engineer, and all bridge openings and culverts carefully fixed there. The profile as platted was inked in, but the grade line was left in pencil. As soon as the openings were fixed, and the soundings noted at the bridge sites and cuts, the estimate of quantities and the cost of each mile were made up by the draftsman and checked by other members of the party; this was then all carefully inked in, and a tracing made. Profiles of final locations were made in 25-mile sections. A portion of such a profile is shown in Plate XIV. This profile and the 5 000-ft. map contain all the information necessary to enable a contractor to bid intelligently on the work, and as all this work was kept up together, it was immediately available on the completion of the surveys.

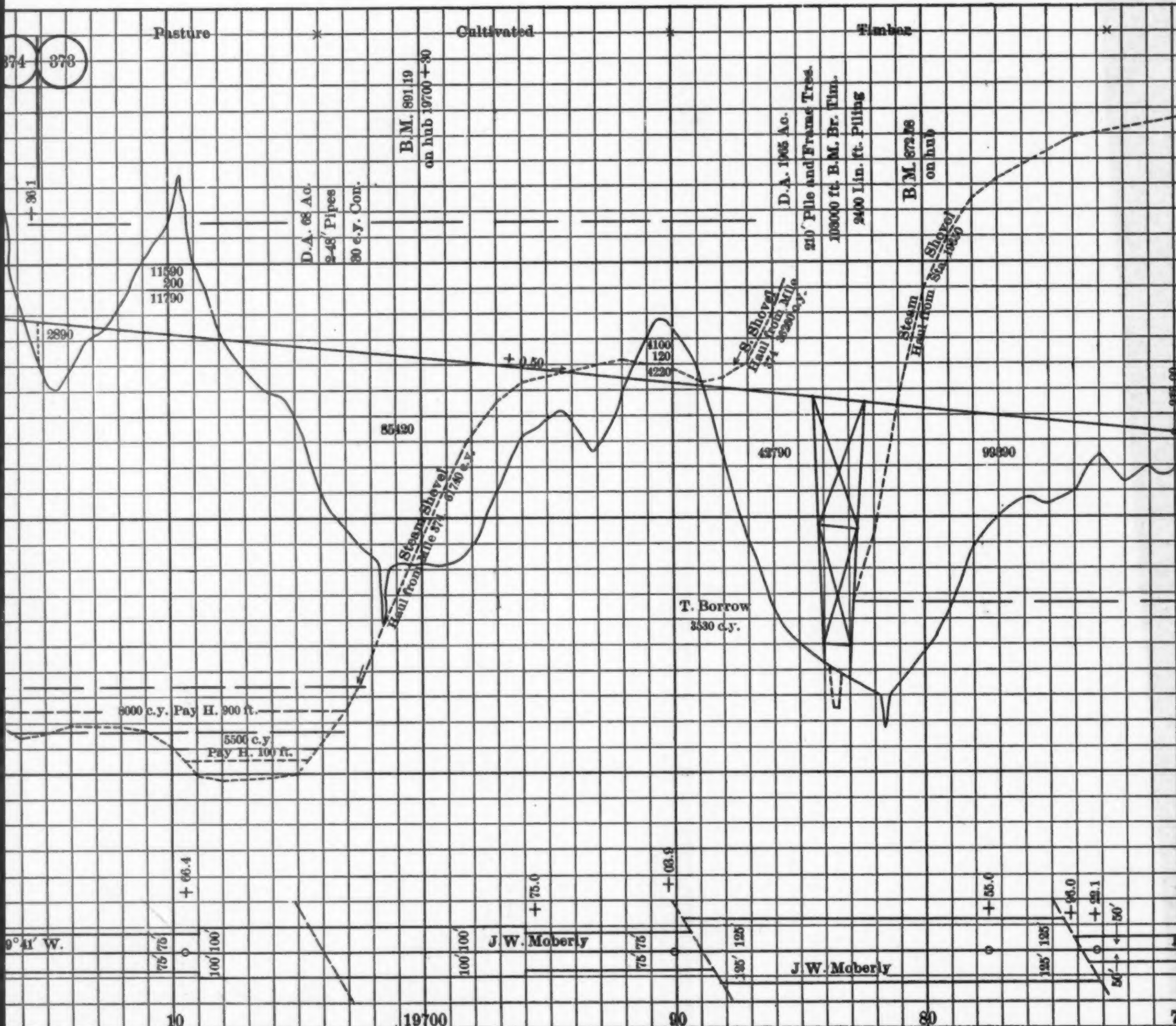
The tracing of the profile of the final location was sent to headquarters as soon as a 25-mile section was completed, together with the right-of-way map and ravine sections covering the same ground.

On receiving notice from headquarters that the grade line, as shown on the tracing profile, had been approved, the grade line on the original was inked in, any changes that were ordered being made, and then this was ready for the Division Engineer having charge of the construction of that section. Reference to the "Instructions to Resident Engineers" on the Choctaw, Oklahoma and Gulf Railroad, by Messrs. Molitor and Beard, will show how the work of construction was co-ordinated with that on the location.

All maps and profiles were carefully lettered in ink on the outside at each end, the lettering running parallel with the axes of the rolls, showing just what they were. On the 400-ft. map, the title might be something as follows:

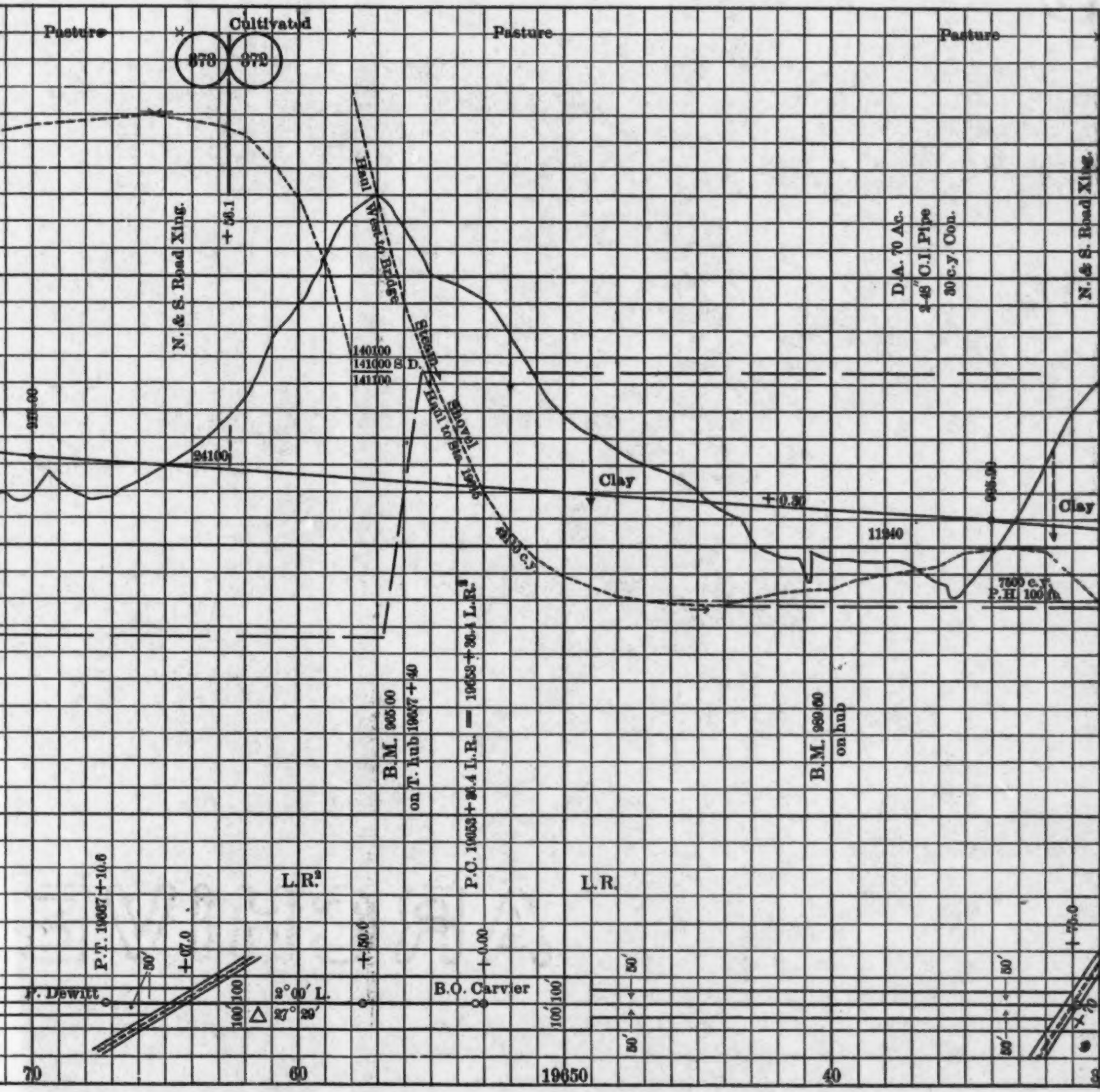


875
874
874
873
Cultivated
Cultivated
Pasture
L.R. + 48.3
954.72
Team Work
7570 c.y. West
− 0.30
105570
1080 C.D.
810 S.D.
107460
D.A. 30 Ac.
D.A. 13 Ac.
D.A. 15 Ac.
24″ Pipe
12 c.y. 2nd Class
Masonry
21050
200 C.D.
150 S.D.
21400
11500
900
11790
D.A. 66 Ac.
2-42″ Pipes
90 c.y. Con.
B.M. 891.19
5190
C. Channel
110 c.y. Exc.
into Emb.
3600
9670
2890
100
100
Haul to Mile 378
Steam Shovel
900
E. & W. Road Xing.
Grade
B.M. 935.80
on hub
B.M. 935.05
on hub 19710 + 40
5000 c.y.
Pay H. 100 ft.
8000 c.y. Pay H. 300 ft.
5500 c.y.
Pay H. 100 ft.
Exc. from C. Channel
into Emb.
85820
Steam Sh
Haul from
P.T. 19767 + 29.0 L.R.² = 19759 + 91.9 L.R.
L.R.
L.R.²
+ 16.0
P.C. 19759 + 90.0
+ 00
+ 30
+ 08
+ 00
+ 62.3
+ 60.0
+ 66.4
Shopes Est.
1° L.
14° 36′
75′ 75′
J.E. May
C.C. Moberly
S. 29° 41′ W.
75′ 75′
100′ 100′
19770
60
19750
40
30
20
10
19700



Pasture
Cultivated
Timber
374
373
+ 36.1
11590
200
11790
2890
D.A. 65 Ac.
2-48' Pipes
30 c.y. Con.
B.M. 391.19
on hub 19700 + 30
+ 0.50
4100
120
4220
85420
Steam Shovel
Haul from Mile 374
S. Shovel
Haul from Mile 374
D.A. 1965 Ac.
210' Pile and Frame Tres.
103000 ft. B.M. Br. Tim.
2400 Lin. ft. Piling
B.M.
on hub
Steam Shovel
42790
99390
T. Borrow
3530 c.y.
8000 c.y. Pay H. 900 ft.
5500 c.y.
Pay H. 100 ft.
+ 66.4
+ 75.0
+ 03.9
+ 55.0
+ 22.1
9°41' W.
75'
100'
J. W. Moberly
125'
50'
10
19700
90
80
70

PLATE XIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 991.
LAVIS ON
METHODS OF RAILROAD LOCATION.



New York - Boston Line,
Providence - New London Section,
John Smith, Locating Engineer.
Final Location Sta. 852 to 1748, Mile 56 to Mile 73

Preliminary Sta.		A.	934	to	1823
"	"	M.	48	"	223
"	"	N.	15	"	329
"	"	O.	0	"	56
"	"	B.B.	17	"	638

The maps accompanying this paper are reproductions of maps actually made in the field, and show more clearly than any written description the kind of work accomplished.

The following is a statement showing in detail the cost of surveys conducted practically in accordance with the methods outlined in this paper. The length of the final located line in this instance was 179 miles, and the work was divided between four parties. The country was similar to that described by the writer, that is, long rolling country, rather badly broken up, the line running across the drainage, necessitating the exploration of a wide range of country on either side of the proposed route. The average quantity of grading per mile was about 100 000 cu. yd., maximum grade, 0.5%, maximum curve, 2°; there was 19% of the line on curve. The writer is especially indebted to Mr. Beard for this information and notes on the same, as well as for much valuable assistance in the preparation of this paper.

Field preliminary expense for 563 miles..........	$14 628.97
" " " per mile..............	25.98
" Location " for 179 miles..........	12 597.92
" " " per mile.......	70.38
Locating Party No. 1: Expense on preliminary, incident to above location and including preliminary and location of 9 miles.................	2 478.02
Office expense charged to above.................	6 446.08
Total cost of preliminary and location 188 miles....	$36 150.99
Total cost per mile........	$192.30

Preliminary Lines.

	Party No. 1. July 5th to October 1st.	Party No. 2. July 22d to October 20th.	Party No. 3. August 1st to November 19th.	Party No. 4. September 21st to October 21st.
	87 days.	90 days	111 days.	30 days.
Miles run and topography taken	145.8	166.3	164.1	23.2
Miles run, no topography taken	39.3	....	16.0	8.6
Total miles preliminary run	185.1	166.3	180.1	31.8
Total number payroll days	1 380	1 824	2 033	635
Average daily number of men	15.9	14.7	18.3	21.2
Average miles per day per party	2.12	1.85	1.62	1.06
Total cost of subsistence	$513.18	$646.42	$763.53	$371.47
Average daily cost, subsistence per man	$0.37	$0.49	$0.48	$0.58
Relative cost percentage to lowest man subsistence	100	133	103	157
Total payroll cost (except teams)	$2 502.55	$2 683.22	$3 381.56	$1 055.55
Average daily pay per man	$1.81	$2.03	$1.66	$1.66
Total cost for teams	$522.00	$560.23	$768.55	$386.15
Daily " " "	$6.00	$6.22	$6.92	$12.87
Contingencies	$88.48	$112.95	$91.84	$125.73
Total cost of party	$3 629.96	$4 002.82	$5 057.96	$1 938.23
Daily " " "	$41.72	$44.48	$45.57	$64.61
" " per man	$2.63	$3.03	$2.49	$3.05
Cost per mile	$19.61	$24.07	$28.08	$60.95
Relative percentage to lowest man per mile	100	123	143	311

Located Lines.

	Party No. 1.	Party No. 2.	Party No. 3.	Parties Nos. 2 and 3 Combined.	Party No. 4.
	65 days.	37 days.	8 days.	48 days.	66 days.
Miles located	56.0	37.8	7.6	42.6	39.2
Total number payroll days	1 400	709	151	1 498	1 283
Average daily number of men	21.5	19.0	19.0	31.2	19.4
Average miles per day per party	0.86	1.02	0.95	0.89	0.65
Total cost subsistence	$515.55	$273.07	$59.50	$599.20	$574.45
Average daily cost subsistence	$0.37	$0.39	$0.39	$0.40	$0.59
Total payroll (except teams)	$2 410.10	$1 143.11	$242.70	$2 582.74	$2 049.25
Average daily pay per man	$1.72	$1.61	$1.61	$1.71	$1.60
Total cost for teams	$434.74	$212.71	$43.10	$496.00	$445.85
Daily " " "	$6.69	$5.75	$5.39	$10.33	$6.76
Contingencies	$143.36	$46.76	$15.70	$196.00	$133.84
Total cost of party	$3 503.75	$1 675.65	$361.00	$3 893.94	$3 203.59
Daily cost of party	$53.90	$45.22	$45.12	$80.29	$48.54
Daily cost per man	$2.50	$2.36	$2.39	$2.57	$2.50
Cost per mile	$62.57	$44.33	$47.50	$90.47	$81.72
Relative percentage to lowest man per mile	141	100	107	204	184

There are various things to be taken into consideration in judging the fluctuations in the cost of these surveys. The preliminary location by Party No. 1 was over a severe country, and embraced the heaviest work on the whole line; at the same time, much difficulty was experienced in getting a grade between certain points on the line located by Party No. 3. Party No. 2 had the lightest country.

There is charged to the expense of Party No. 4, the cost of moving a long distance from other work to this line, which amount, together with the short time they were engaged on preliminary, abnormally increased the cost of their work; at the same time, it is evident that this was decidedly the most expensive party on the work, their work per unit of cost, costing more.

For instance, their subsistence was 57% more than that of Party No. 1, and the team hire more than double that of the other parties; while the actual number of men in the field was relatively the same. It is probable that the cost of the work done by this party was really about 60% more than the others instead of 200% as shown by the cost per mile.

On location, Party No. 1 carried a very heavy and expensive sounding party, consisting of a man in charge, four or five laborers and a team; the nature of this work was such that it was much more expensive than that conducted by any of the other parties.

After completing the location, Party No. 1 was engaged in running other preliminaries and locating a short branch, the cost of this work not being distributed, but included in the total cost of the survey, the amount being $2 478.02.

On location, Parties Nos. 2 and 3 were combined after each had run in a short distance separately; this was necessitated by the approaching cold weather and the desire to complete the location at the earliest possible moment; the result shows it to have been an uneconomical proposition as far as cost per mile is concerned; but both of these parties had much additional preliminary work to perform as they proceeded with the location.

What has been noted of Party No. 4 on preliminary is true on location, though its cost is somewhat burdened by the charges incident to moving the party elsewhere, and the fact of its happening about Christmas, when many men were given vacations with pay. This Christmas expense was encountered to a somewhat less extent by

every one of the parties, and tended to increase the total cost, but, taken as to Parties Nos. 1, 2 and 3, the statement is a fair average of what a thorough survey under like conditions will cost.

Besides the organization, as noted on page 111, which was practically the same as that engaged on these surveys, with the exception that there was only one topographer, there was the expense of an expert, at $150 per month and his expenses, engaged in an examination of the country adjacent to the line, for the purpose of determining the quantity of sand and stone available for construction purposes. The amount of this expense is $545.84.

In all this work it was considered absolutely necessary that all parts of it should be kept up together, and, with the large party available, it was found feasible to assign the men so that any part of the work which lagged behind could be brought up to date; the weak point is, of course, with the leveler on preliminary, if he gets behind there is little to do but to wait until he catches up. It is necessary, therefore, that an especially good man should be selected for this position. Physical ability to hustle is absolutely necessary, and the rodman must expect to trot between stations and the instrumentman between set-ups, if they cannot keep up by walking.

The leveler in a party should be, not only accurate, but quick. As an instance of what can be accomplished: On one of the lines referred to, starting from the west, more than 100 miles of preliminary were run to the eastern end of the line in 20 working days (not including Sundays and moving camp). On one day, the leveler covered 8 miles. On returning and making the final location, when every care was taken to have the levels as accurate as possible, equalization of sights being insisted on, and there being ample time for the leveler to do the work properly, no variation from any bench-mark was found greater than $\frac{5}{10}$ ft., the final check on the bench at the western end being about $\frac{3}{10}$ ft.

In making the preliminary location, or rather, the writer would prefer to say, in running the preliminary lines, he considers that the result to be obtained should be regarded more in the nature of making a topographical map of a strip of country through which the final location will pass, and through which runs a sufficiently accurate base line or lines, than in running a line which will be very close to the final location. There is only one place, in his opinion, to adjust the final location, and that is on a good topographical map.

This, of course, will not be misunderstood as relieving the Locating Engineer of the necessity of running these preliminary lines with judgment and a good idea of their relation to the located line. All the good judgment and "eye for country," relied on so much by some of the older locating engineers, are still as necessary as ever, but they must be supplemented by scientific methods and hard work.

The statement in regard to the final adjustment will possibly evoke some discussion from the many men who have saved thousands of dollars by slightly changing a curve in the field or otherwise after the final location is made, and the writer will admit, of course, that there is hardly a line located to-day, or likely to be, where every foot of it is exactly where it ought to be, but, in anything but the most minor changes, he believes that the fault will invariably be found in the fact that the original topographical map was not correct, or the projection not well made.

Provided the topography is generally correct, which it should be, to be of any use at all, it is possible to project a line on it, which will be the best line the country affords, and, if the work is properly done, this line can be laid out on the ground. In adjusting the line to the topography, the line can be changed and a profile obtained fifteen times on the map while it is being changed once on the ground, and all the problems affected by the change studied.

The writer is well aware, of course, that the practice as outlined in this paper will necessarily be subject to many modifications to meet different conditions.

In conducting surveys in tropical countries or in other places where it is difficult to obtain experienced engineers, and then only at largely increased salaries (in tropical countries about two to three times as much as is noted in this paper), other methods become necessary, but the writer believes that the same ends should be striven for. In these cases, there is a great temptation to the engineer in charge of such work to shrink from the responsibility of insisting that he be given *carte blanche* by his employers in the matter of engaging such assistance as he may need and in the payment to them of adequate salaries.

There is much mountain country where transportation is extremely difficult, where everything possible must be done to lighten the equipment, and where a great deal of reconnaissance can be done in more or less detail with a light party, either by a separate party ahead of a larger one, or before a larger party is organized and put in the field.

Such a party can do much good work with a transit used as a level, or by using the stadia, in eliminating certain entirely impractical lines. In any event, under such conditions, both parties should be controlled by the same man, as what might have been regarded as impractical under one set of conditions may become entirely so under others; all this, however, is matter which will suggest itself to the experienced locator.

There are probably many old locating engineers, many who have done excellent work with much less equipment and fewer men, who will hold up their hands in horror against such an organization and equipment as is outlined here, but railroads themselves, as they exist to-day, are all the evidence necessary to prove that other methods than those of the past are necessary to meet changed conditions. Scientific methods must be applied to the conduct of location, as well as to the design of bridges, terminals, locomotives, etc.; in fact, on a proper location or otherwise the future of the railroad is almost entirely dependent.

In submitting this paper to the consideration of the Society, the writer does not wish to be understood as advocating any hard and fast rules for railroad location. No two lines are alike, topography is never the same, and nothing will take the place of experience, good judgment, and much hard work. He knows there are many good locating engineers who entertain different ideas, and he hopes they will submit them for consideration. He does firmly believe, however, that it will most certainly pay in the long run to obtain in every case, whatever the method may be, at least as much information as was obtained on the work described, as shown by the maps and profiles accompanying this paper.

DISCUSSION.

E. SHERMAN GOULD, M. AM. SOC. C. E.—From the point of view of practical railroad engineering, the speaker is inclined to rate this paper very highly. Mr. Gould.

The proper location of a railroad is a matter of the greatest importance, for it affects, not only the immediate cost of construction, but that of operation and maintenance for all time, and any material change in the location of a road in active use is most difficult and expensive. In the words of the late Arthur Mellen Wellington, M. Am. Soc. C. E., the skill of the railroad engineer is shown by the imposing bridges, the high embankments, deep cuts and long tunnels which are not to be found on his line. Certainly, in skilful hands, material can be taken out more cheaply with transit and level than with the steam shovel.

The first thing in the location of a railroad is the collection of data by means of a survey; the inferences and conclusions to be drawn therefrom come later. The survey cannot be too complete, and Mr. Lavis well says that in his experience the completeness of the survey is apt to vary directly as the completeness of the outfit, and proceeds to describe such an outfit as he has found necessary to successful work under given conditions. Many of the minutiæ may excite a smile from some of the "old war horses," as indicating in their opinion, a too great preoccupation with the refinements of civilized life, but there can be no doubt that more and better work can be done in the field, and with a heartier spirit, when these little comforts are supplied.

Mr. Lavis begins at the beginning, with a list of the camp equipment furnished by the road with which he was connected. In glancing over this list the only point calling for criticism which met the speaker's eye is the German silver table service. He would entirely condemn the use of this metal for any use which brings it in contact with food.

All the camp regulations described seem to be sound and intelligent. Evidently, they were efficacious in securing that very difficult desideratum when in camp, namely, an early start. Nothing is said about it, but doubtless there were rules regarding the occasional airing of bedding, particularly as the men were required to make up their cots for the night immediately on rising, or at least before going to breakfast. Casually, the difficulty of procuring fresh meat and vegetables is mentioned. By a fortunate provision of Nature, salt meats suit the taste of those living in the open air much better than fresh, for a steady diet, and the same may be said in regard to the absence of milk and even sugar in one's tea and coffee. The

Mr. Gould. palate seems to crave, in such conditions, the simplest and crudest flavors.

Naturally, the principal interest centers upon the engineering rather than the culinary features of the paper. The desiderata in the present case, or the objects for which the survey was undertaken, are enumerated under five headings, the second one containing the kernel of the whole proposition, namely:

"To be sure that the line obtained was such that no other line could be built through the same country with the same or better ruling grades, with less expenditure, at the same unit prices."

This sentence very neatly expresses the object of all such surveys.

Perhaps the most important feature of the paper, and the one most likely to provoke discussion, is the insistence with which the author dwells upon the superiority of a paper location over a field location, that is to say, the taking of the ground into the office, and locating the line there, more or less mathematically, rather than doing it entirely by eye, upon the ground. If the speaker is not mistaken, this idea was carried to the limit in the case of the Lyons and Geneva Railroad, in France and Switzerland, by the construction of a plan in relief, reproducing in miniature the whole stretch of intervening country over which the road must pass, and locating the line upon it. The increasing effort to bring the field work into the office in all branches of engineering is a marked characteristic of the present trend of practice in America. While this may degenerate into "easy-chair engineering," when carried to excess, the speaker is inclined to believe that, in the case of the final location of a railroad, or other line, the getting of full details of the topography laid down upon a contour map is the true way to secure the best results. One point, however, has perhaps not been sufficiently dwelt upon in this paper, which is, that after the line has been thus located on paper and transferred to the ground, a final revision is absolutely necessary, by the best locating talent in the outfit, which will probably result in changes of greater or less importance. Indeed, it appears to the speaker that the whole process of scientific location, from the first examination of the territory as shown on the atlas map, down to setting the final slope stakes, is one of "trial and error," from field to office, and from office back again to the field. It also appears to him that, except in those cases where there is almost no choice to be exercised, the skill and experience of the locating engineer are best shown in the rapid comprehension of the broad features of the problem, rather than in elaboration of details. The qualities which enable an engineer to take this almost instinctive grasp of the possibilities of a wide and varied stretch of territory seem to be akin to those which characterize a great military commander in his selection of strategic points. The gift may be greatly

cultivated, but, for its fullest power, it must be inherent in the individual. It is also the opinion of the speaker that the value of this gift may be over-rated, and that when time allows—and if the surveys be promptly commenced instead of being relegated to the last moment, time generally will allow—no amount of "eye for the country" can supersede the patient and exhaustive instrumental examination of all the possibilities of the territory. At any rate, it is certainly safe to give to younger members the advice not to rely upon the fancied possession of this gift, but rather to cultivate the eye and judgment by painstaking study and observation. Mr. Gould.

In the present case, the necessity of exhaustive preliminary lines, previous to final location, was evidently realized, for Mr. Lavis says:

"Instructions were given the locating engineers to spend all the time necessary on investigations, to be sure they had the best line through the country traversed before putting in the location, on the ground."

It is certainly vain to cross-section a strip 800 ft. wide on both sides of a radically vicious line, when a 5-mile swing to the right or left would open up a practicable route through easy country. That is to say, it is useless to try to make the best of a bad line, when a good one is ready to the hand.

In regard to the instrumental work, the running out of all tangents to an intersection is sound practice, unless a curve is finally adopted which sweeps in several changes of direction, when the total deflection between the points of curvature becomes the true angle of intersection. In such case it might not be necessary to produce the tangents to an apex, but, in all cases, as the author states was done, the position of the P. C. and P. T. should be calculated and the hubs set from the apex, or by long chords or otherwise, before the curve is swung in, even at the cost of much additional labor. When the curve is run on final location, if a satisfactory connection is not made with the previously established P. T., the transit should be set up upon it and the curve run backward until the discrepancy is worked out, or the existence of an error is detected.

No mention of easement curves seems to have been made in the paper.

The speaker is free to confess that he does not like the form of transit notes used in this survey. His own invariable practice has been to have two columns for deflections, left and right, and to carry the balance over from page to page. In this way, by applying the balance to the first course of the survey, the calculated course can be checked at any given station. Distances should be added up and carried forward in a similar way, so as to check them with the numbering of the hubs. In reading angles, both verniers should be

Mr. Gould. read, when, if the graduation of the limb runs from 0° to 180°, as is usual, the sum of the two readings should always be 180°, thus affording a perfect check upon the recorded angles.

Mr. Thompson. WILFORD A. THOMPSON, JUN. AM. SOC. C. E. (by letter).—The writer desires to ask Mr. Lavis why the plane-table is not used in sketching topography for railroad location? Of course, many engineers think the plane-table is very inaccurate and slow, but the writer has used this instrument, in work for the United States Geological Survey, in the West, and has run lines ranging in length from 1 to 15 miles in a day, at the same time sketching the topography for ½ mile on each side of the line. These surveys were on a scale of 1 in 90 000, with a 20-ft. contour interval, and the writer is sure that, with a plane-table, a topographer can make a more accurate map in the field, where he can see the country, than can the draftsman from the best of notebook sketches.

With the stations on the line to give the lengths of base lines, and from the elevations given by the levelman, a very accurate map of a section 2 000 ft. wide can be made almost as rapidly as the levelman can cover the ground.

Starting at a given station and orienting on the line, short lines should be drawn to all houses, lone trees, fence corners or other objects which can be recognized from some of the next succeeding stations, such points being chosen as will be useful in locating the topography. In passing to the next station the instrumentman should note carefully the topography passed over, and after setting up should intersect objects at which shots were taken at previous stations. Distances can be scaled from the map of the intersected points, and their elevation can be found very accurately by the vertical angles. In almost any country a sufficient number of points can be located to indicate the drainage, the "backbone of topography." Having located these points and ascertained their elevations, it is a simple matter to sketch in the intervening topography.

The chief advantage is that when the topographer returns to camp the map is ready for use, and there is no necessity for waiting for hours while the draftsman works up an inaccurate map from the notes.

A rapid method of finding the elevations of intersected points is what is called the "stepping-up method." Knowing that the interval between the cross-hairs is proportional to the distance, then if the distance to a point scales 550 ft. from the instrument the interval intercepted by the cross-hairs is 5.5 ft. The alidade is leveled and the point where the upper cross-hair strikes the ground is noted. The elevation of this point, then, is one-half the cross-hair interval (very closely), or 2.25 ft. above the instrument. Then

elevate the telescope until the lower cross-hair intersects the point formerly covered by the upper cross-hair. A number of these steps may be taken. If the upper cross-hair coincides with the ground at the intersected point after two steps, then the elevation at that point is two and one-half times the cross-hair interval, or 13.25 ft. above the instrument. Then, adding the height of instrument gives the difference in elevation between the ground at the plane-table and at the intersected point. This method is subject to a slight error, but, for short distances, it is far more rapid and accurate than pacing and using a hand-level. Even if there is an error of 1 ft. in the elevation of a point 1 000 ft. from the line, it is within the limit of error of a location survey, and is closer than a hand-level elevation. The writer has run lines 10 miles long, using a stadia rod for distances and reading the elevations to the nearest foot by the foregoing method and has closed without error. Mr. Thompson.

With a long bubble on the telescope, and with cross-hairs close together, or an extra cross-hair between the middle and upper cross-hairs, the writer is sure that the plane-table would be most useful on location surveys. Such a table should be about 22 by 24 or 30 in., and the tripod should be light. On the under side at each end there should be rollers to carry a strip of drawing paper, 20 in. wide. If the line runs too close to the edge of the paper a new point can be taken near the other edge.

S. Whinery, M. Am. Soc. C. E. (by letter).—The method of locating railroads described in this paper is quite different from that in which some of us had the good fortune to "assist" (with an axe or a flag-pole), in our younger days. The up-to-date way is only scientific; the old way belonged to a higher order—it savored of Genius. The new way accomplishes results by plodding and platting; the old one reached conclusions with something of the swiftness and certainty of Instinct. The new locating engineer is merely an educated ordinary individual, and is compelled to work things out; the old one, like the poet, was born, not made—he simply came, saw, and conquered. Given, a correct topographical plat of a hillside, a set of curve-models and other paraphernalia, any of us with a reasonable share of ability can locate on the plat a curve that will best fit the topography, and, later, can transfer the curve to the ground. But, disdaining plats, to approach the hillside boldly with your line in hand, to take in at a glance the P. C., and the degree of a curve that will fit the hill comfortably, and, at the proper point on that curve to sail off on a tangent that will strike the bull's-eye 2 miles away, that was the work of a Genius. Even if it was sometimes necessary to "cut and try" a little, the line "got there" with celerity and no little success. Mr. Whinery.

If this is an exaggerated picture, it must be admitted that there

Mr. Whinery. is more than a grain of truth in the chaff. The earlier locating engineer who attained eminence in his calling had, undoubtedly, a peculiar natural aptitude for his work. His "organ" of locality was large. He was a born woodsman, and never got "lost." His natural faculties were highly trained, because he depended upon them and not on any such auxiliary devices as are now used. His "off-hand" judgment of topography, and the grades and curves that would fit it, was more acute and accurate than that of his degenerate successor, because the latter is not compelled to rely so largely upon his unaided faculties.

All this may be admitted, and yet it can be maintained successfully that, upon the whole, the new method is better than the old. It reduces the work to a system, and the results, if less brilliant, are more certain and reliable, and at least equally expeditious.

While details may differ, the paper describes well the approved modern method of railroad location, and the fullness of details makes it a very valuable contribution to the literature of the subject. Conditions, of course, may modify the procedure. The lines, the location of which is described in the paper, seem to have been, from the start, destined for construction; roads were to be built between certain points, and the only question was to find as quickly as possible the best route and location. If, as often occurs, preliminary lines were to have been run out for the purpose of determining the practicability of the route for a railroad, the building and even the final location of which, later, depended upon the success of the promoters in financing the enterprise, the methods and means would doubtless have been somewhat modified. The outfit would not have been so complete. The promoters would have wanted, to show, a reasonably good line and a profile that would at least fairly represent, if not minimize, the difficulties to be overcome, and their demand could not well be ignored.

But, assuming that the office of the preliminary line is mainly, as it should be, to determine distances and elevations, and to furnish a base from which to obtain correct topography, the appearance of the preliminary profile may well be disregarded.

In running such preliminary lines, the most important and responsible position in the party, next to the Chief, is that of topographer, and it is also the most difficult to fill. The work of the linemen and the draftsman, while, of course, it must be accurate, requires no special ability; but the topographer must be an expert at his work. He must possess, in a large degree, the peculiar gifts of the old locating engineer, must have a keen eye and a good judgment for locality, distance and elevation. If he depends too much on the tape-line and the hand-level, and lacks discrimination as to the relative importance of topographical features, he will neither

be able to keep up with the party nor do his work satisfactorily. Mr. Whinery.
Particularly must he have the ability, natural or acquired from experience, to judge of the relative importance of the topography he sketches. He must know at a glance, from the general lay of the country, that the final location will hug this hillside closely, and its topography, therefore, must be taken accurately, while that other will not be touched, and, therefore, may be sketched with less care. A poor topographer is somewhat worse than useless, while a superior one is cheap at almost any salary.

There seems to be little to criticise in the conduct of the work described by the author. Not many engineers have been as fortunate, in having employers who appreciated the value and economy of complete equipment and liberal provision for the comfort of the men. The absence, from the equipment, of the medicine chest is noticeable. The writer has always considered it of importance to have along with the party a small supply of simple or standard remedies and surgical appliances, which every intelligent person knows how to use.

In running out the final location line, some modifications of the method described in the paper have been found advantageous in the writer's work. They are not new, and may not be novel, but he has not seen them described in print, and they may be of some interest to other engineers.

In the summer of 1880 the writer had charge of the preliminary and location surveys for the northern half of the New Orleans and North Eastern Railroad, his work extending from Meridian, Miss., southward about 100 miles. With some alternative routes, about 120 miles of located line was put on the ground. The preliminary surveys were made with care, especial attention being given to accuracy of instrument work and chaining, and to taking the topography. The lines crossed the drainage system of the country diagonally. The streams had flat and rather wide valleys, separated by dividing ridges from 80 to 150 ft. high, with rather abrupt and broken slopes. No special effort was made to get good preliminary profiles, the object being, while keeping the line reasonably near the probable located line, to run the preliminary where it could be gotten through easiest, or would best serve the purpose of the topographer. The preliminary lines were carefully platted by the "Latitude and Departure" method.

In running the final location line, the object sought was to do as much of the work as possible in the office tent, confining the work of the party largely to putting in the stakes on the ground, from notes furnished to the transitman (who had charge of the party).

The method of procedure was as follows: A tentative location line was put on the map as carefully as possible, and its profile was

Mr. Whinery. platted on the preliminary profile from the topography. Very full notes were made of the relation of the located line to the preliminary, such as intersections, distances at right angles from the preliminary line, etc., using the actual and projected stations on the two lines. With these notes and the profile, a tape-line and hand-level, the writer, with an assistant, then went over the ground, noting at every controlling or important point the correspondence of the projected profile with the ground, and any changes that seemed necessary or desirable. Returning to camp, the projected location was carefully revised in accordance with the notes thus taken. This done, the courses, curves, etc., of the revised line were computed or measured with great care, and a full and accurate description made out for the use of the transitman, including very full reference notes to points on the preliminary. All this work was kept well ahead of the transit party. As long as the location line conformed to the reference notes, with relation to the preliminary line, the transitman had nothing to do but stake out the projected line. But if any divergence was noted, it was his duty to introduce such possible modifications as would maintain the relative positions of the two lines. Thus, after running out a tangent, say, 1 500 ft. long, to a projected P. C., if he found this P. C., came ahead of, or short of the preliminary station opposite which it should be, he moved the P. C. to its proper position and began his curve. If he found himself nearer to or further from the preliminary line than he should be, he could usually so modify his P. C. and his curve-angle as to throw the succeeding curve where it should be. Fortunately, the transitman was a very competent man, and handled these small corrections very skillfully.

The results were very satisfactory, and the located line was carried forward rapidly. Occasionally, when the profile of the day's work was made up, it would not be satisfactory, and part of the line would have to be re-run the next day; but this going back was so unusual that it came to be considered by the transit party as a reproach upon the office work. The average progress made was somewhat more than 2 miles per day, which, considering that the country was mostly covered with timber and underbrush, and the hill slopes rough, was regarded as very good work. The accuracy with which the located line could be projected was sometimes surprising. At one place there was a 7-mile tangent, and as it all lay through a timbered and brushy country, special care was taken at its beginning to lay its course accurately, so as to avoid the possibility of having to re-run it. Whether due to careful work or to good luck, it came out within 20 ft. of the point intended, and this difference was readily compensated for in the succeeding curve.

The fact that all the curves were begun and ended with spirals made the work a little more complicated than usual. (As far as the

writer knows, this was the first railroad in the country to be originally located with spiralized curves.) Mr. Whinery.

The noteworthy variation from ordinary practice in this work was the careful review, on the ground, of the projected location, and revision before the line was actually staked out, a practice which proved so advantageous that it is strongly recommended.

The same general method was used successfully, under very different conditions, in the location, a few years later, of the "Standard Gauge" Railroad, up Lookout Mountain, at Chattanooga. Here, the line ascended the rough and steep sides of the mountain to an elevation of about 1 300 ft., on a grade of 3½%, absolutely continuous, except as modified by compensation for curvature. In this case the preliminary line was plotted on a scale of 100 ft. to 1 in.

As a rule, both money and time may be saved in the preliminary surveys by careful and thorough reconnaissance, which should be made by the engineer who is to be in charge of the work, before the party is started. Equipped with such land-office and other maps as are obtainable, a skeleton map on a scale of 2 in. to the mile for sketching, and the usual outfit of pocket instruments (a good aneroid barometer, a detached thermometer, a pocket compass, and a pedometer which can be set to read miles and fractions for various lengths of step of man and horse), the best probable line can usually be determined definitely before the party starts. The use of these pocket instruments is not always as well understood and as fully availed of as it should be. This is particularly true of the aneroid. So much has been written on the proper use of this invaluable instrument that it need not be repeated here. The writer has always read the inch-scale rather than the altitude scale, using for reduction to altitude, a table of "height in English feet of a column of air corresponding to one-tenth of an inch in the barometer at different temperatures and elevations." This table may be found in most treatises on hypsometry,* but it is not usually sufficiently extended for convenient use. A more extended table (Table 1), which will be found sufficient for the engineer's ordinary purposes, is appended. To illustrate its use, suppose the following observations are taken:

At the lower station, Barometer 29.20 in., Thermometer 55°
At the upper station, " 29.00 " " 50°
The tabular number corresponding to the first reading is..........94.5
The tabular number corresponding to the second reading is..........94.1

Average..........94.3

*See, for instance, "Smithsonian Meteorological and Physical Tables," by Prof. Guyot.

Mr. Whinery. This is the average elevation corresponding to a difference of 0.1 in. in the barometer, and this multiplied by the difference in barometer readings, two-tenths, gives $94.3 \times 2 = 188.6$ ft., for the difference in elevation of the points of observation. While not absolutely correct, this method is more nearly so than any other equally simple. To overcome the principal source of trouble in the use of the barometer—its normal changes with the weather—the writer has used one or two simple and fairly satisfactory methods. These changes are usually quite regular and progressive. If, with the barometer hung up, readings be taken when the user first wakes up in the morning, and again after breakfast when he is ready to start out, and similarly when he stops an hour at noon for lunch and rest, some idea of the rate of change can be formed, and applied to correct the forenoon and afternoon observations in the field. For further and final correction, at the end of the trip, the records of the nearest weather bureau station may be applied, or, better still, those of a special barometer left at a permanent station and read by an assistant every hour of the day during the period.

It is rather common to sneer at the value of the barometer for such reconnaissance work, but when used with proper care, it is a most useful and reasonably reliable instrument. In a reconnaissance made by the writer, extending over about ten days, in the mountains of North Carolina, three aneroids were carried by the party, and at all important points the mean of their readings was used. For final corrections the mean of the barometer records of the weather offices at Knoxville and Atlanta was used. The results, when tested by actual levels, were remarkably satisfactory. The pedometer, used with judgment based on experience, is also a most valuable auxiliary on reconnaissance. Good reconnaissance work can only be done on foot or on horseback, but, in the latter case, provision must be made for having the horse led "around" on a considerable part of the trip. Except for a hasty trip to get the general lay of the country, a vehicle is useless; and fairly good maps may usually take the place of such a trip.

The author's remarks on the importance of care and deliberation in working out the best possible final location are pertinent. It may be safely asserted that there are in this country a surprisingly large number of railroads located so badly that, were it now possible, a competent engineer could afford to re-locate, at his own expense, the whole line, taking for his compensation a small percentage of what he could save in cost of construction and operation. But while the engineer is not infrequently to blame for faulty location, it is generally attributable to undue pressure upon him to hasten the work and to avoid expense. Very few employers, even among the abler and better informed class of railroad projectors,

appreciate the great economy and importance of careful and conscientious location, or have any adequate idea of the time and cost it involves. Mr. Whinery.

TABLE 1.—Height, in Feet, of a Column of Air Corresponding To a Tenth of An Inch in The Barometer at Different Temperatures and Elevations.

Barometer Reading in inches.	Temperature of the Air, in Degrees, Fahrenheit, Exposed Thermometer.																
	15°	20°	25°	30°	35°	40°	45°	50°	55°	60°	65°	70°	75°	80°	85°	90°	95°
27.8..	90.9	91.9	93.0	94.0	95.1	96.1	97.2	98.2	99.3	100.3	101.4	102.4	103.5	104.5	105.6	106.6	107.7
27.9..	90.6	91.6	92.7	93.7	94.8	95.8	96.9	97.9	99.0	100.0	101.0	102.1	103.1	104.2	105.2	106.3	107.3
28.0..	90.2	91.2	92.3	93.3	94.4	95.4	96.5	97.5	98.6	99.6	100.6	101.7	102.7	103.8	104.8	105.9	106.9
28.1..	89.9	90.9	92.0	93.0	94.1	95.1	96.2	97.2	98.3	99.3	100.3	101.4	102.4	103.4	104.5	105.5	106.5
28.2..	89.6	90.6	91.7	92.7	93.7	94.8	95.8	96.8	97.9	98.9	99.9	101.0	102.0	103.0	104.1	105.1	106.1
28.3..	89.3	90.3	91.4	92.4	93.4	94.5	95.5	96.5	97.6	98.6	99.6	100.6	101.7	102.7	103.7	104.7	105.8
28.4..	89.0	90.0	91.1	92.1	93.1	94.1	95.1	96.1	97.2	98.2	99.2	100.2	101.3	102.3	103.3	104.3	105.4
28.5..	88.6	89.6	90.7	91.7	92.7	93.8	94.8	95.8	96.9	97.9	98.9	99.9	101.0	102.0	103.0	104.0	105.0
28.6..	88.3	89.2	90.4	91.4	92.4	93.4	94.4	95.5	96.5	97.5	98.5	99.5	100.6	101.6	102.6	103.6	104.6
28.7..	88.0	89.0	90.1	91.1	92.1	93.1	94.1	95.2	96.2	97.2	98.2	99.2	100.2	101.2	102.2	103.2	104.2
28.8..	87.8	88.8	89.8	90.8	91.8	92.8	93.8	94.8	95.8	96.8	97.8	98.8	99.8	100.8	101.8	102.8	103.8
28.9..	87.4	88.4	89.4	90.4	91.4	92.5	93.5	94.5	95.5	96.5	97.5	98.5	99.5	100.5	101.5	102.5	103.5
29.0..	87.0	88.0	89.1	90.1	91.1	92.1	93.1	94.1	95.1	96.2	97.2	98.2	99.2	100.2	101.2	102.2	103.2
29.1..	86.8	87.8	88.8	89.8	90.8	91.8	92.8	93.8	94.8	95.9	96.9	97.9	98.9	99.9	100.9	101.9	102.9
29.2..	86.5	87.5	88.5	89.5	90.5	91.5	92.5	93.5	94.5	95.5	96.5	97.5	98.5	99.5	100.5	101.5	102.5
29.3..	86.2	87.2	88.2	89.2	90.2	91.2	92.2	93.2	94.2	95.2	96.2	97.2	98.2	99.2	100.2	101.2	102.2
29.4..	85.9	86.9	87.9	88.9	89.9	90.9	91.9	92.9	93.9	94.8	95.8	96.8	97.8	98.8	99.8	100.8	101.8
29.5..	85.6	86.6	87.6	88.6	89.6	90.6	91.6	92.6	93.6	94.5	95.5	96.5	97.5	98.5	99.5	100.5	101.5
29.6..	85.3	86.3	87.3	88.3	89.3	90.3	91.3	92.2	93.2	94.2	95.2	96.2	97.2	98.2	99.1	100.1	101.1
29.7..	85.1	86.1	87.0	88.0	89.0	90.0	91.0	91.9	92.9	93.9	94.9	95.9	96.9	97.9	98.8	99.8	100.8
29.8..	84.8	85.8	86.7	87.7	88.7	89.7	90.6	91.6	92.6	93.6	94.5	95.5	96.5	97.5	98.5	99.4	100.4
29.9..	84.5	85.5	86.4	87.4	88.4	89.4	90.3	91.3	92.3	93.3	94.2	95.2	96.2	97.2	98.2	99.1	100.1
30.0..	84.2	85.2	86.1	87.1	88.1	89.1	90.0	91.0	92.0	92.9	93.9	94.9	95.9	96.8	97.8	98.8	99.7

As a result, very few locating engineers can look back with satisfaction on a good deal of the work which they felt compelled, under pressure of time and expense, to do less carefully than their judgment dictated.

C. P. Howard, Assoc. M. Am. Soc. C. E. (by letter).—The writer has read Mr. Lavis' paper with much interest, it being the only paper he can recall which treats so fully this important subject. Though Mr. Lavis deals with the details of location rather than general principles, they are details which affect very vitally the general results. Mr. Howard.

The plan of allowing a locating engineer and an assistant locating engineer, besides instrumentmen, to each party seems to be a good one for a thorough preliminary line, which is really a topographic survey. The locating engineer can pick out the general route, while the assistant stays with the party and examines the

Mr. Howard. ground immediately ahead of the transit. The transitman should not have to leave his instrument. Of course, where the general route has been well established by a thorough reconnaissance, or where, on account of topographic features, it is necessarily circumscribed, one locating engineer for each party would probably be sufficient. The kind of country traversed may make some difference as to the proper organization of parties and methods of survey. Surveys have been conducted in flat country with little, or no, topography; and sometimes very approximate methods have been followed in rough country by engineers of good reputation; but it would appear to be best to adopt, as a general rule, that method which is best suited to rough country, which means an accurate contour survey. Mr. Lavis' plan of survey, in general, seems to be correct. There are certain points, however, on which the writer takes issue with him.

The first of these relates to the importance of accuracy, which Mr. Lavis seems to underrate. This is illustrated by the method he advocates of "running in" the location. Instead of running his transit line by the notes of the projection, he takes a hand-level in the field and tests various elevations to see that they fit his projected profile, and then, presumably, varies the line to fit the elevations. It would seem to be much better, as a general rule, to take correct topography, and then locate the line on the ground to correspond with the projected line on the map. If the projection goes outside of the limits of accurate topography, run more preliminary line, or take new topography. However, in order to put down a projected location on the ground accurately and satisfactorily, all work must be well done, and the scale of the map must be large enough to admit of accurate work. In such case the engineer can go ahead and run in his line with confidence, without stopping to compare profiles, shift his line, or test elevations. Of course, no rigid rule need be drawn, and a good engineer, trying to locate "to a profile," with somewhat imperfect projection and topography, may at times satisfactorily revise and shift his line in the field; but the plan of accurate topography, and a line on the map correctly reproduced on the ground, would appear to be the better method. Slight adjustments of grade or line can be left to the resident or construction engineer after he has calculated and balanced his quantities. In the writer's opinion, the whole plan of a projected location reproduced on the ground depends, for its success, speed and economy, on the accuracy of the work. The importance of this increases with the ruggedness of the country. In the mountains, the actual expenses of the physical work of running a transit line and levels are very much increased. Therefore, it is important to avoid the extra work of running tangents to intersection, and to

Mr. Howard.

run the line in the right place the first time. After toiling up and down, over cliffs and ravines, clearing through heavy timber and thickets, it is very disheartening to find the line 10 or 20 ft. off, and a great temptation to let it go as it is. It should also be borne in mind that, within certain reasonable limits, work can be done about as fast accurately as inaccurately. As to the preliminary or topographic survey, there should be an accurate transit or angle line, accurate levels, an accurate plat, and correct topography. All are important, but the last, though depending more or less on the others, is the most important.

An accurate transit line does not necessarily mean measuring to a pencil mark, but both chainmen should carry plumb-bobs to be used when necessary. The writer will not discuss here the details of leveling, but, as to the plat, he would suggest the following:

Get the best tools. First, the drawing board should have a steel straight-edge, let into and screwed down on one edge. Use an adjustable steel tee-square, sliding along the straight-edge of the board, with the blade representing the meridian or east and west line, and a metal protractor (Brown and Sharpe, or Crozet) applied to the blade of the tee-square. This method of platting, in the writer's opinion, is not only the most accurate, but the quickest. In rough or mountainous country, plat the line on a scale of 100 ft. to the inch; in rolling country, 200 ft. to the inch; in flat country, 400 ft. to the inch. Plat the center line on sheets of tough white drawing paper, about 13 by 18 in. in size, and ink it in with red ink. The sheets, of course, should lap over each other and be matched by suitable cross-marks, etc.

A meridian line should be drawn in black ink on each sheet, and the whole platting should be carefully checked, as to both bearings and distances. The utmost mechanical accuracy is desirable.

The sheets for one day's work, having been platted, are taken into the field by the topographer the next day. The topographer carries with him a light, hollow drawing board. This board is made of two thin boards or leaves, about 16 by 20 in. and $\frac{3}{8}$ in. thick, fastened to solid sticks on the edges, leaving a hollow space of about $\frac{1}{4}$ in. inside, and with the stick at one end hinged so as to open and shut. This little board, not much bigger than a slate, serves the topographer both as a drawing board, and as a case for the sheets not in use.

For taking topography, the method recommended by the writer is essentially the same as that of Mr. Lavis, differing only in certain details. There should be one topographer for each party, and in a country where land lines are frequent or important, one or two men should be detailed especially for that work, as suggested by Mr. Lavis. In the topography party proper, there should be one

Mr. Howard. topographer, two hand-levelmen and two rodmen. Each hand-levelman carries a book in his pocket, or in his left hand, and records the contours. His hand-level is fastened on a 5-ft. stick. The rod is 10 ft. long, and is divided every half foot, with the foot marks painted in large figures, beginning with 0 at the center of the rod.

The advantage of the 5-ft. stick and the 10-ft. rod and its numbering is evident. If the elevation of the station is 518, the first contour to be taken on the upper side is 520. The hand-level on its 5-ft. stick is held at the station, and the rodman, measuring the distance with his rod as he goes, goes up the hill until the hand-levelman reads 2 on the rod (2 ft. below the center of the rod). The rod is then 2 ft. higher than the center. The rodman calls out the distance from the center line, and the hand-leveller jots down in his book the contour and distance. Next, if the ground is still rising, the hand-levelman proceeds up the hill with level and stick until he can just read the top of the rod. He is then at Contour 525; and the rodman measures up to him as before, calling out the total distance from the center stake. The rodman then continues to measure up the hill until the hand-level reads the bottom of his rod. The rodman is then at Contour 530.

By this method each contour is located on the ground on each side of every station as far out as necessary. The hand-leveller keeps his notes of contours and distances in fractional form like ordinary cross-sections. The plus at which a contour crosses the center line is found in the same way. At each station the hand-levelmen call out to the topographer from the books the distances of the contours from the center line at their last station, and he locates the points by scale on his sheet. By the time his men have cross-sectioned one station, he should have drawn in on his sheet the contour lines up to the preceding station. One good topographer, with two hand-levelmen and two rodmen, will generally keep up with a transit party. The breaking up of the whole corps for one day at each camp to help out the topography party, as suggested by Mr. Lavis, is to be avoided if possible.

As a general thing, all distances of contour lines from the center line should be measured by the rodman with his rod. On hillsides it is a convenient method, and even on flat ground a rodman can measure distances quite rapidly with a little practice, turning the rod over end for end as he goes. Occasionally, a tape should be used for measuring distances, but, if used to any extent, one or more extra men should be added to help out. This is the case when the line is in the center of a bottom, with a creek on one side and the foot of a hill on the other, the distances to both of which are to be accurately determined. In a rolling or flat country, where an error of 10 ft. in distance would not be material, the distance of the con-

tours from the center line can be stepped, as recommended by Mr. Lavis; and in a still flatter country the location of the contours beyond the distance stepped may be estimated, for such information as they may give; but they should be recorded in the book in brackets, and only dotted on the sheets. Mr. Howard.

Contours, in general, should be taken for 200 or 300 ft. on each side, where the ground is flat, and sometimes farther; or to an elevation of 25 or 30 ft. above or below the center line on steep ground. On rough ground they should be taken on each side at every 100-ft. station. On flat ground they can be taken at every second or third station. It is desirable, of course, to have the preliminary line run as nearly as possible on the ground to be occupied by the location, as the topography becomes less accurate the farther it is from the center line; and the more ground to be covered sideways, the less will be covered lengthways, along the center line, in the course of a day's work.

Topography parties should work somewhat slowly in the beginning, until they can avoid mixing contours, and have formed habits of accuracy. They can then begin to develop speed. It is of the greatest importance to get the work started right. The writer knows of a recent case, in a mountainous country, in which, on account of faulty topography, and other inaccuracies, the work of one or two parties for two or three months had to be thrown away, and the work done over again.

All topography is drawn in on the sheets in the field during the day and inked in at night. These sheets are the map. They should not be allowed to get wet, consequently, a little time is sometimes lost in drizzling weather; or, until the rain stops, topography can be taken in a book and afterward transferred; but, in any case, the topographer cannot do much work in the rain.

In general, there is no advantage in taking topography in a book. The contour lines can be drawn in more accurately on the sheets in the field, and there is no difficulty in keeping the sheets reasonably neat and clean. Besides, it is quite an undertaking to transfer topography from a book to a map. But if the engineer likes maps in large rolls, he has only the trouble of replatting the center line, as the topography can be transferred more rapidly from the sheets to a map than from a book to the map. The sheets constitute a contour map and detailed record of the results of the survey, and should be carefully preserved.

On these sheets the location is projected. After a satisfactory projection has been made, both as to line and profile, the next step is to take off the notes for running in the location on the ground. The same mechanical accuracy is necessary as in platting the line. There should be noted the position of each P. C., P. T., and P. C. C.,

Mr. Howard. at right angles from the proper station and plus on the preliminary line; the bearing of each tangent, by protractor, which should be the calculated bearing of the tangent as run in on the ground; the central angle of each curve, being the difference between the bearings of tangents; and the length of each tangent and curve; the last, when measured on the projection, being a check on the central angles. Then the location should be run in continuously from one end. A small correction will usually be made at the beginning of each curve, or of each curve and tangent, to compensate for the error up to that point and make the line check out ahead; but no part of the line will be run over again unless the divergence from the projection is great enough to be material. For small differences, say in general less than 5 ft., it will not be necessary to re-run the line; as it is generally difficult to determine, on location, before the line has been carefully cross-sectioned, and quantities accurately balanced, whether or not such a shift will hurt the line. In flat country a divergence of more than 5 ft. from the projection is generally allowable. The correction at the beginning of each curve is the simplest. A blank stake is driven at the point designated for the P. C. by the notes of the projection. If the tangent misses this point by say 4 ft., multiplying the cotangent of the central angle of the curve to be run by 4 ft. gives the distance that the P. C. must be moved backward or forward on the tangent so as to check out on the tangent ahead. Corrections at the beginning of tangents can also be made very readily by a transitman who is good at figures by slightly changing the length of the curve. This, however, changes the bearing of the tangent and the length of the curve ahead, and requires a little more figuring. The same may be said of a correction at a P. C. C. on a compound curve. It is very important that the calculated bearings of the location should check with those of the preliminary survey. If there is a difference of say 15 minutes between the calculated bearings of the preliminary and the location lines the transitman will find his line continually diverging from his projected line in one direction. Consequently, he should "angle" across to a preliminary hub occasionally, to check his calculated bearings by the calculated bearings of the preliminary; and if there is a difference, he should, for running purposes, correct the bearings of the location to check with those of the preliminary. The position, or "ties," of every P. C., P. C. C. and P. T. should be carefully noted on the right-hand page of the transit book, with reference to the preliminary line and at right angles thereto, thus:

85+92.3 P. T...............(24 L 94+76 prelim.).

This information is necessary, in order to draw the actual location on the sheets.

In the writer's opinion, as a general rule, the method of running tangents to an intersection should not be used. It is attended with difficulties, and is often impracticable in the mountains. The intersection point is likely to be across a mountain, or on the other side of a river, if not in the middle of the stream. The object to be attained by the whole method is that the engineer can select and lay out the best location on paper, both as to line and profile; and, by his notes of projection, run the same line continuously on the ground from one end to the other in the shortest possible time, and with nearly the same accuracy as if the whole had been calculated by latitudes and departures. Mr. Howard.

EMILE LOW, M. AM. SOC. C. E. (by letter).—The writer has had considerable experience in railroad location, especially on the Mexican Central; Pittsburg, McKeesport and Youghiogheny; Pennsylvania; Schuylkill Valley, and the Norfolk and Western Railroads. Methods similar to those outlined by Mr. Lavis were used on these roads. Mr. Low.

The writer, however, desires to take issue with the author for his preference for roll maps, and agrees with Wellington, that separate sheets have many advantages, especially in camp, where the drafting facilities are not always of the best. The so-called protractor sheets are found to be very convenient in this respect, the writer preferring the 19 by 24-in. sheets, on Weston's Linen Record paper, which is a commercial size. He also prefers to orient the sheets, the edges being kept absolutely north and south, or east and west, as may prove most practicable.

For field use, these sheets have much to commend them. They can be readily carried in a convenient wooden portfolio, which also answers as a drawing board. On the roads mentioned, the scale generally used was 200 ft. to 1 in., so that a sheet covered about a mile of line.

For office use, these sheet maps were traced in sections of convenient length, or, in cases where the line was more or less direct, an entire division could be shown in one tracing. In this way the advantages of both sheet maps and roll maps could be combined.

Mr. Whinery aptly makes this statement: "It may be safely asserted that there are in this country a surprisingly large number of railroads located so badly that * * *."

In another place he also makes this statement: "The up-to-date way is only scientific; the old way belonged to a higher order—it savored of Genius."

It is far from the writer's intention to make any invidious comparisons, but he would venture the assertion that many of these badly located railroads can be traced to the Geniuses mentioned, and the present generation of engineers is kept busy correcting the mistakes of a former one.

Mr. Low. The writer has had considerable experience in the location of railroads, both by the "practiced eye" and by the more scientific methods of topographical maps. In the location of the Pittsburg Southern Railroad, now the Pittsburg-Wheeling Division of the Baltimore and Ohio Railroad, the former method was used. A preliminary line had been run, and it was the intention to plot this, and project a location in the ordinary way, as is now common. As the contract for the road had been let, and the contractors were waiting to start work, with not a location stake driven, heroic measures had to be adopted. The line crossed a number of summits, and, therefore, it was decided to plot only these parts, project enough location to cross them, and run in the remainder by the "practiced eye."

Much of the line was on a maximum grade, and on these portions grade stakes were set ahead, by means of the level and the chain, for suitable distances. This could be done readily, as the line generally passed over cleared and grazing lands. A location was then fitted to the stakes thus set, and it was surprising how, after a few days' practice, the required curvature could be guessed at.

In the valleys, this procedure was not needed, and the line was run as the judgment of the locating engineer indicated. This judgment was sometimes warped by trying to avoid artificial as well as natural objects, in fact, in one instance the location was influenced by avoiding some favorite apple trees, which, unfortunately, were in the way; the line being deflected to avoid them. At other times, growing fields of corn would limit an extended vision ahead, but, from a convenient tree, a sufficient altitude could generally be attained to make a survey of the conditions ahead.

Notwithstanding these limitations, good progress was made, and if the writer remembers aright, a stretch of 14 miles, through a very hilly country, was located in a few weeks, the party covering themselves with glory by having made the maximum number of miles per day, regardless of all other considerations.

It would be gross flattery to say that the best possible alignment was obtained by such methods, but still the road, when completed, was good enough to whisk trains over at 30 miles or more per hour.

One feature, not touched in the discussion, relates to location: not whether the best location on a certain grade or alignment has been obtained, but whether or not the railroad has been built in the best locality. The following is an example: The former Pittsburg and Connellsville Railroad (now the Pittsburg Division of the Baltimore and Ohio Railroad) was located and built by the writer's father, Sigismund Low, C. E., under the direction of the late Benjamin H. Latrobe. At Pittsburg, Pa., between the city

terminal proper and the suburb now known as Glenwood, there was a choice of location, along the foot hills bordering the Monongahela River, or along the river bank itself. The writer's father strongly advocated the river bank line, but his selection was over-ruled, and the line built at a higher elevation, back from the shore. Since then, part of the line has been changed to the first suggested location. At other points rival railroads which control these have been built. For another long distance, large manufacturing plants have sprung up, and these now prohibit the occupation of this land by a railroad. Mr. Low.

Had the line been built originally along the river, inestimable advantages, in the control of the river front, would have accrued to the present company, and these cannot now be procured, except at the cost of millions of dollars. Of course, the immense developments which have taken place, since the original construction, could hardly have been foreseen, but, still, some of them might have been anticipated. It is likely, in this case, that the controlling factor in adopting this particular location was cheaper first cost.

For some years the writer was locating engineer on the Nescopeck Railroad, a branch line of the Pennsylvania Railroad in the anthracite coal regions. One of the features of this railroad was the location of a grade line, north of Rock Glen, Pa., down Black Creek to Nescopeck Creek. The original location was made on a 1% compensated grade. Subsequently, this was changed to 1.1, 1.2, 1.3, 1.4% grades, and, finally, to a 1.5% grade. Each line was a perfect one, as far as location went, but the steepest one was selected, because the construction was the cheapest.

The writer was also associated with the late A. M. Wellington, M. Am. Soc. C. E., as Locating Engineer on the Guanajuato Branch of the Mexican Central Railroad, of which his father was in charge. As Guanajuato is off the main line, it had to be reached by a branch, and several junction points were suggested, among others, the cities of Irapuato and Silao. The line to the former was the longer, but that to Silao crossed a range of hills. A tunnel line had been located, but was rejected for this reason, and a line over the hills was ordered. The most feasible one, up the thalweg of the valley, on the western slope, was first located, but had a 2½% maximum grade, but with very light work, a very direct alignment, and no curvature of any moment. This was considered too steep, and an easier grade line was ordered, against which the writer's father offered strenuous objections, as being impracticable. The writer was delegated to locate this line. The valley mentioned was bordered by very steep slopes, cut up considerably by gullies. The lighter grade, with a high compensation, soon brought the line along the steep slopes, and, to avoid heavy work, had to wind in and out

Mr. Low. among the ravines. The consequence was that the alignment was a succession of maximum curves, with light grades rising higher and higher above the valley, and when the main valley was reached the line was several hundred feet above it, and had to be deflected at right angles, entering convenient side valleys and gradually coming down to the level of the main valley. The length of the line was nearly double that of the steeper alignment, with no compensating advantages. It is needless to say that the steeper line was built.

In referring to the instances mentioned, the writer intends to convey the idea that, in the location of a railroad line, common sense, coupled with judgment, is also a requisite.

Mr. Oakley. F. T. OAKLEY, M. AM. SOC. C. E. (by letter).—The writer was very much pleased to read this paper, for, while he believes that most of the items treated in it are elementary and to many locating engineers are as a-b-c's, they are worthy of record, on account of the author's reason, that there is little of public record of these matters, and on account of the fact that the details of methods of location are of such great importance in obtaining the results sought.

A description of methods which have now the sanction of many years of trial should be appreciated, not only by students of location, but by those whose practice has been largely in the line of maintenance, and those who are charged with the responsibility of employing locating engineers, and it is hoped that all will have a better appreciation of the value of those details of the work which are not a part of theory.

The importance of the commissary department of a locating party can hardly be over-estimated. The writer wishes to suggest the use of a lunch-can, of galvanized iron, in the form of a cylinder, with a tight-fitting lid. Such a can should have trays, about 5 in. deep, which will fit snugly in the can, the trays having handles which will fold inside of the tray. One or two of these trays may be subdivided to provide compartments for small articles. Such cans will be found to be much more sanitary than boxes.

The writer is also of the opinion that a small medicine chest, containing a few simple remedies, would be a valuable addition to such an outfit, and especially useful in a sparsely settled country.

With reference to the drafting table, he would suggest that a board, strengthened by cleats, be used for the top, this board to be supported by a "saw-buck," so constructed that one set of legs will fold between the other set, the outer set being connected by two cross-pieces, one at the top and the other at the bottom on the opposite side of the legs. The inner pair of legs should have cross and X-braces, the two sets of legs being held together by bolts at the point of crossing. When this "saw-buck" is open, chains may be placed across the top at each end, secured at one end of the chain

by a screw-eye, and at the other by a screw-hook. The table may be raised and lowered by the adjustment of these chains. Mr. Oakley.

In the writer's experience, the work of running preliminary lines may be very much expedited, and at the same time greater accuracy be secured, by the following procedure with the transit party:

When a "hub" has been set and the transitman has been called up, the engineer in immediate charge of the party can direct the chainmen where to go, and keep them in very good line until a few stakes have been set, when experienced chainmen can then maintain their own line for some distance across open country, and trimming and cutting can proceed, together with the chaining through timbered country. In this manner, by the time the transitman arrives at the "hub" and has his instrument set up, the chain party will be well out on the new tangent. The transitman, having his vernier set at zero, should then line up on the stakes already set, and, continuing, keep the chainmen in line. The chainmen should proceed to the point where the locater indicates that another "hub" is required and a change of direction is to be made. At this point the chainmen, having secured line as for a stake, will put in a "hub," drive a tack or nail in it, and place the line pole upon the tack just set. The transitman will then turn his instrument to this point, only a slight adjustment of the instrument being necessary therefor. When he signals the chainman "all right" the chainman answers the signal "all right, come ahead." The transitman at this time should turn to the back-sight and read the angle.

The object of this procedure will be apparent to most persons: First, it keeps the chain gang moving, and thereby expedites the movements of the entire party, no time being lost in waiting for the transitman or for the back flagman. Second, it gives an accurate measurement of the angle, without the necessity of repeating the measurement, the course observation, in most cases, being a sufficient check upon the angle reading. Of course, the angle reading may be repeated if desirable. The angle will be measured by this operation immediately after the sights have been taken, thus eliminating any error due to the instrument standing for some time after being set up. Those who have run a transit over unstable ground, or when frost is coming out, will appreciate this method. The elimination of such errors will be found to make it possible to check more closely the angles of various lines, and be very valuable in platting more than one line over the same territory. Consequently, it will expedite the work of the draftsman upon the map. The exact position of the stakes on preliminary lines is not of material importance.

The writer has been accustomed to keeping level notes in a

Mr. Oakley.

TABLE 2. —(*Continued.*)

Spiral Table.—Length of Standard Sub-chord = 30 ft.

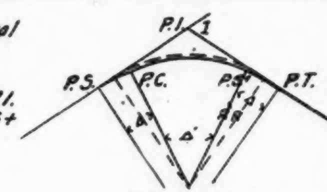
D	L.	S.	N.	Δ.	C.	Co-Ordinates.		Long Chords.	Radius.	D.
1	30'	.01'	1	09'	15.00'	.026	30.000	30.000	5,729.64	1
2	60'	.05'	2	36'	30.00'	.209	59.999	60.000	2,864.88	2
3	90	.18'	3	1°21'	45.00'	.706	89.995	90.000	1,909.90	3
4	120'	.42'	4	2°24'	60.00'	1.674	119.979	119.990	1,432.27	4
5	150'	.82'	5	3°45'	75.04'	3.270	149.936	149.970	1,145.46	5
6	180'	1.41'	6	5°24'	90.08'	5.650	179.841	179.930	953.96	6
7	210'	2.25'	7	7°21'	105.18'	8.968	209.657	209.740	816.77	7
8	240'	3.34'	8	9°36'	120.35'	13.376	239.331	239.627	713.44	8
9	270'	4.77'	9	12°09'	135.67'	19.023	268.795	269.470	632.50	9
10	300'	6.54'	10	15°00'	151.14'	26.052	297.960	299.110	567.15	10

Let D = Degree of curve to be spiralized.
I = Total angle of intersection. Δ = Central angle of each spiral
Δ' = Central angle of circular part of spiralized curve.
N = No. of chords in spiral. Z = No. of any chord point.
L = Length of spiral. Standard chord in spirals = 30 ft.
C = Amount to be added to tangent to set P.S. and P.T. from P.I.
S = Shift of circular curve. Ex. secant of spiralized curve = S + Ex. secant of circular curve.

$I = \Delta' + 2\Delta$. $\Delta = 9D^2$ $L = D \times 30$.

$S = \dfrac{8L^2D}{1100000}$ R = Radius of circular curve.

A = Radius of spiralized curve. A − S = R.



DIAGRAM

For discussion of principles upon which these Tables are based see "Railroad Spirals" by D. M. Green.

Mr. Oakley.

TABLE 2.

Table of Deflections, P.S. to P.T. Chord = 30 Ft.

D	1.	2	3	4	5	6	7	8	9	10	Deflections in diagonal column are used in running Spiral from P.S. to P.C.
1	06′										03′
2	15′	24′									12′
3	24′	42′	54′								27′
4	33′	1°00′	1°21′	1°36′							48′
5	42′	1°18′	1°48′	2°12′	2°30′						1° 15′
6	51′	1°36′	2°15′	2°48′	3°15′	3°36′					1° 48′
7	1°00′	1°54′	2°42′	3°24′	4°00′	4°30′	4°54′				2° 27′
8	1°09′	2°12′	3°09′	4°00′	4°45′	5°24′	5°57′	6°24′			3°12′
9	1°18′	2°30′	3°36′	4°36′	5°30′	6°18′	7°00′	7°36′	8°06′		4° 03′
10	1°27′	2°48′	4°03′	5°12′	6°15′	7°12′	8°03′	8°48′	9°27′	10°00′	5°

EXPLANATION OF TABLE.

Look in column D for degree of curve which it is proposed to spiralize. The first deflection on that line will be the deflection from tangent at P.S. for chord point 1; the second for point 2 and so on untill the number of points set will = D. The last one will be the P.T.

Number the chord points from P.S. to P.C. begining with 0 at P.S. also from P.S. to P.T. begining with 0 at P.S.

TO CONNECT TWO TANGENTS WITH A CURVE, USING SPIRALS.—Set instrument at the P.I. and measure angle = I. Take Tangent corresponding to this angle from table of tangents to 1° curve. Divide by "D" (= degree of curve); to quotient add correction in Column C, and measure this distance along each Tangent to set P.S. and P.T. Move instrument to P.S. and to set any point on Spiral we have ($z^2 \times 3'$) = deflection from Tangent. To set P.C. turn from Tangent $\frac{\Delta}{3}$. To turn Tangent at P.C., back sight on P.S. and turn $\frac{2\Delta}{3}$, calculate deflections for shortened radius, and put in Circular Curve. Central Δ of Circular Curve = $I - 2\Delta$. Move instrument to P.S. to set any point on Spiral we have ($D \times 9z - 3z^2$) = deflection in minutes from Tangent at P.S. to any point on Spiral. The last deflection will be $\frac{2\Delta}{3}$. Move instrument to P.T. and back sight on P.S., turn $\frac{\Delta}{3}$ and instrument will indicate succeeding Tangent.

Mr. Oakley. slightly different manner from that ordinarily given in textbooks. He uses the first column for stations, in which are recorded also the initials for turning points, bench-marks, and height of instrument. In the second column is placed, first, the elevation of the bench-mark, and, immediately under this, not necessarily upon a line, the back-sight with its proper minus sign. Under this, drawing a line and performing the subtraction, the height of instrument is obtained. The third column contains only the rod readings, and the fourth column the elevations to be entered in the profile. When a turning point is desired, the rod reading upon this point is placed in the second column, the proper plus sign for this reading should not be omitted, the process of adding then is performed, obtaining the elevation of the turning point. The object of doing the work in this manner is to perform the arithmetical operations, upon which the elevations depend, so that there is the least chance for error, and it seems to the writer that the ordinary method of doing such work, placing one set of figures immediately under the other, is the easiest and most conducive to accuracy. This method has the advantage of leaving more columns upon the page for other notes, and keeping only elevations for profile work in one column. It also keeps the figures of the rod readings contiguous to the height of instrument, so that the operation of subtraction is easy and can be performed much more quickly than where these figures are placed across the page. As the author states, the rodman should keep and check the heights of instrument and turning points.

The writer notes that the author says nothing about the use of easement curves. The engineers probably did not find it necessary or desirable to use easement curves where the maximum curvature was so very light. This is not the case, however, on all railway lines, and it seems that such a paper, not being confined to the methods used upon one railway system, should not omit the question of the use of easement curves. Table 2* was used by the writer some sixteen years ago, when the use of easement curves was not as common as now; and only a few were published. He has found that this table has the advantage of being very concise; at the same time, it is very easy to use in the field with the transit. Blue prints may be pasted in the back of field books and always be accessible, so that little calculation is required.

Experience with drafting done in the field inclines the writer to favor the use of narrow rolls of paper. He has used paper not more than 18 in. wide, but would prefer 21 or even 30 in., 21 in. being the half of a 42-in. roll, is a convenient size to carry in a camp chest. Unless the lines are very sinuous, it is not difficult to keep

* Published in *The Railroad and Engineering Journal* in 189C, by Franklin Riffle, M. Am. Soc. C. E.

them on narrow paper for long distances, if care is used in starting the map. Mr. Oakley.

It is also found to be advantageous to use contours for all topography taken near the line, which is done with considerable care, for distances ranging from 200 to 500 ft. on each side of the line; beyond this limit, sketches should be made of prominent and important features. For instance, the location of a hill or a divide in the neighborhood of the line, when not given by accurate topography, may be shown by hatch lines, or some method may be used to indicate approximate elevations, or, where no elevations are attempted, the presence of abrupt changes in contour may be simply indicated.

Too great care can hardly be exercised in selecting the topographer. He should be an untiring worker, and have an eye for country, otherwise many important features will not receive the care they should, and time will be wasted on unimportant ones. The writer remembers when the topographer was a boy who could take slopes, and was chosen because he was nimble enough to keep up with the party. Such a policy will surely result in inaccuracy of topography, and may make a poor location of a line which otherwise should be a good one.

Experience has thoroughly demonstrated that it is false economy to put upon the locating engineer the burden of locating; also, sometimes directing the instrumentmen, and frequently looking after the draftsmen and estimators; and it is noted with some satisfaction that there is a tendency toward greater liberality in the assistance given the locating engineer, allowing him more freedom of movement and consequently greater opportunity to study more thoroughly the problems of location. This indicates a better appreciation of the value of well located lines by those in authority.

The writer recalls an instance of considerable difference in the cost of maintaining a locating party on account of the long distance it was necessary to transport supplies. In one instance, the cost of maintaining the party, per man per week, was \$5.61 for board, and, when the cost of transporting supplies to camp was deducted, this amount became \$4.73. In this particular instance, the supplies were hauled about 125 miles, and it was necessary to maintain two teams almost constantly for this work. The same party in a different locality cost about \$2.30 per week per man. These figures include the wages of the cook, but not of the teamsters.

O. H. Tripp, Assoc. M. Am. Soc. C. E. (by letter).—Perhaps a word on camp equipage and the management of a party in a region different from that described by Mr. Lavis may be of interest to the younger members of the Society. Mr. Tripp.

In the older portions of Maine it has not been customary for

Mr. Tripp surveying parties to live in camp, as hotels or farm-houses can nearly always be reached, and the expense is less than that involved in running a camp. However, in recent years most of the railroad work has been in places where camping is absolutely necessary, but where such an outfit as Mr. Lavis describes would be out of the question. It is very rarely practicable for the party to ride to or from work, and, on some of the surveys for the Bangor and Aroostook Railroad, as well as on many of the preliminaries for the Canadian Pacific Railway, it was necessary to move camp without the aid of teams. With this class of work, however, the writer can claim no acquaintance.

Parties in Maine require a great quantity of bedding in cold weather, and the blankets and spreads for a party of fifteen, rolled up and ready for moving, would surprise anyone not posted in such work.

Tents of drilling with flies of sheeting answer, but are short lived and are of little value after one season if the season has been a bad one. As far as the writer's experience goes, wall tents are always used, and the size, of course, varies with the ideas of the chief engineer. Personally, the writer does not believe in putting more than four men in a tent. This arrangement permits the use of small tents, and, in addition, prevents much noise and confusion, which are apt to occur when more men are quartered together. A tent 10 ft. square (a "four-breadth" tent) with 3 or 3½-ft. walls will accommodate four men properly, and the work required in pitching such a tent is much less than for one 12 or 14 ft. square. "Headquarters" tent should be five breadths long, as it must have room for the drafting table, which is generally in two pieces, so as to pack in small compass, and is supported by crotch-stakes and cross-pieces cut at each camp. This tent usually accommodates the chief of party, with the transitman, leveler, and rodman.

The subject of mosquitoes and kindred "vermin" is too important to be passed over without mention. An opinion prevails among many that in the higher latitudes mosquitoes are much less troublesome than in warmer climates. Of course, the lower latitudes have their special pests, from which the latitudes of Maine are exempt, but, as far as the mosquito is concerned, the fact seems to be that (within wide limits) the higher the latitude the worse the pest during the season of its activity. This season begins in May, and the writer has been badly bitten in September, though they are past their worst by that time. Years ago, when the writer first went into the northern woods on railroad work, he was told by one of the axemen that mosquitoes "came on snow shoes and went on skates," and his first season's experience convinced him that there was much less exaggeration in the state-

ment than would at first appear. Plenty of mosquito netting and plenty of small safety pins properly used will keep this particular pest away (though not out of hearing) during the hours of darkness, but they will find their way under the screen as soon as the light gets strong (and it must be remembered that it gets light early in latitude 46° in June). Mr. Tripp.

The minge (the Indian's "bite 'em no see 'em") cannot be kept out by anything short of cheese-cloth, but, fortunately, as a rule, this insect does not work all night.

The black fly is perhaps the worst enemy through the day, in a country where he thrives. A few places in Maine seem to be particularly infested by this pest, and the worst experience the writer ever had was in the valley of Pleasant River (in the region of the Katahdin Iron Works). In the late spring of 1881 the writer was engaged there as transitman on a preliminary survey. At that time it was the custom to wear knee boots on surveys in the woods, and for days it seemed as though the writer's bootlegs were full of flies. Oil-brushing did not seem to do much toward keeping them away. The fly bites out a small piece of skin and when he goes away leaves the blood running. The bite is poisonous, and, if let alone, frequently causes a running sore. Instances have been known where men who did not know how to protect themselves have been laid up for days from the effect of bites. As far as the writer knows, there is nothing which will keep them entirely away. By using the best of the many preparations it is possible to live among them, but, without using anything, it is out of the question. For a man using an instrument (or for similar work), the best protection for the hands and wrists is a pair of light-weight oil-tanned gloves, with the tips of the fingers cut off and a piece of thin leather sewed into the opening in the palm. The leg of a thin stocking (or enough of it to reach at least to the elbow) should be sewed to the wrist of the glove. This will be serviceable in stopping vermin of all kinds. For oil there is nothing better than castor oil, with oil of pennyroyal (2 or 3 oz. of the latter to 1 pint of the former), and with enough oil of tar to give the mixture a good clear mahogany color. On going out in the morning, this should be applied to the face and neck freely and frequently, until a coating is formed on the skin; after that an application every half hour through the day usually answers the purpose.

No party can be comfortable after September 1st (and some years not nearly as late as that) without stoves. These are of the pattern of a small "air-tight" but with top, bottom and sides of sheet iron. The pipe should run diagonally out of the front of the tent, for, if allowed to go up through the roof, the tent will be set on fire in a short time by falling sparks.

Mr. Tripp. The cook tent, of course, must be of larger size, depending on circumstances, as it must have room for supplies, and, in cold weather, serve as a dining room. Some parties have a cook stove, and some have not. Bread is usually baked in a "reflector," in front of an open fire, and the beans are generally cooked in the ground.

The bed is commonly made somewhat after this style: A "foot-log," from 6 to 9 in. through and about as long as the width of the tent, is laid down at the proper distance from the closed end of the tent, and the straw, hay or boughs, as the case may be, put down in the space thus enclosed, then should come a rubber sheet large enough to cover the whole berth, then some cheap blankets doubled and wadded with cotton. For covering, each man should be furnished with a pair of good woolen blankets. Upon the advent of cold weather these are commonly made into a bag, by folding them into half their width and then sewing them across the foot and about two-thirds the way up the open side. This gives flaps to fold closely around the shoulders, and allows each man to have one, two or three thicknesses over him, as the weather may demand. Over all is laid a "spread" large enough to cover the whole berth, with ample allowance for "tucking in." This spread should be of two thicknesses of the "spreading" used by lumbermen, and heavily wadded with cotton. This outfit will allow men to sleep comfortably when the outside temperature is —20°, as the writer can testify from experience.

It is not customary to provide a table for dining purposes, though of course it is very desirable when means of transportation will permit. Dishes are usually of tin, and spoons of the same (or similar) material.

When there is sufficient daylight, parties are supposed to work 10 hours on the line, which, of course, sometimes means a day of 12 or 13 hours from camp. Dinner is carried to the party, and, of course, it is necessary to reduce the weight of dishes, etc., as much as possible.

The writer's experience forces him to dissent from Mr. Gould's views in regard to the use of salt provisions, no party with which the writer has ever been connected having been for any length of time at all satisfied without fresh meat; and it is unnecessary to suggest that it never pays to have any feeling of dissatisfaction among the men, if it can be avoided, as it soon shows in the work, and very plainly. Of course, there are many times when fresh meat and vegetables cannot be had, and canned goods, although a poor substitute, are a great help. Milk, as well as fresh meat and vegetables, should be provided when practicable; in fact, the writer believes that the fare in camp should be, as nearly as possible, like good home fare, for with such fare the men will do more work and do it much more cheerfully.

Mr. Tripp.

The party is made up of chief of party, transitman, leveler, rodman, two chainmen, back-sight man, and from three to five axemen. The practice of adding a topographer, etc., is on the increase. The camp force is made up of commissary, cook and "cookee" (or cook's helper), and it is needless to say that on their faithfulness and ability the comfort and, in a great measure, the efficiency of the whole party depend. It is not always easy, however, to get good men, the woods cook too frequently being actuated apparently by a desire to see how much lard he can possibly put in everything he cooks.

On moving day (which may vary from once to four or five times a week) a team is procured from the most convenient place, the men pack up their personal belongings before leaving camp in the morning, and, generally, the tents are up, beds made and everything in its proper place when the party reaches the new camp at night.

Methods of work are as varied as the men who have charge, but it should always be remembered that methods which are advisable in one region may be utterly out of place (in fact entirely impracticable) in other regions. In the northern portion of Maine the reconnaissance must be made on foot, and usually through forest or, what is much worse, through the brush and debris left by lumbering operations, or, worst of all, through and over the all-too-frequent "blow-down," and the engineer must bear in mind the fact that a slip in his idea of the country may mean, not only days, but weeks, of a party's time later on; and, with the best man and all the pains he can take, such slips sometimes occur. Of course, no one moves without the ever-present pocket compass; this, with a hand-level and a small aneroid, is about all the instrumental outfit that is of use ordinarily, and but little dependence can be placed on the aneroid unless another is kept in camp and the records compared.

As to the preliminary survey, it is probable that sufficient time is never given to it, as the chief engineer is generally hurried, from the time the first stake is driven until the last rail is laid, and, of course, the effects are to be seen forever after. There can be no doubt as to the advisability of making the map of the country traversed as full and complete as practicable—and the more complete the better—but, at the same time, the writer is a firm believer in the (sometimes sneered at) "eye for country" which has been mentioned in this discussion. This, however, would be more in evidence in exploring than afterward. The plane-table might be of great service for topographical work in some regions, but the attempt to use it in a densely wooded region would seem somewhat like the much-talked-of "horizontal stadia," and not worth while.

The transit used should be of such size as not to wear out the transitman in carrying it—a 5-in. limb is ample, and, in many (if

Mr. Tripp. not most) cases, for preliminary work, one of the smaller instruments, with a 4-in. limb, might well be used. Whatever its size, it should be furnished with a level attachment as well as with stadia hairs and vertical arc. This is essential for the taking of side slopes and the location of such buildings, lot corners, etc., as may be within range in open ground.

It will be understood, of course, that in Maine there is no uniform system of land division, as in the West, hence the opportunity of locating one's self by lot corners is to a great extent lacking. The northern townships were laid out with sides on the true meridian (±) while the older settled parts were laid out to suit the occasion, and when the lines were called north and south it meant by needle at the time of their location. The plans of the townships are usually procured before making surveys, and are of more or less value, in proportion to their accuracy.

Among the minor but important things that tend to expedite the work the writer would suggest that at least one of the axemen should be furnished with a bush-hook, in addition to the axe, and that a supply of machetes should be provided for use in swamps and wherever small stuff in any quantity is to be encountered. More than double the rate of progress will be made in small growth if the party is thus provided. Nothing is more disheartening to axemen than to work for days at a time where there is nothing big enough to stand up to meet a blow of the axe.

The plans for locating purposes are usually made on a 400-ft. scale, but, in places where that scale does not show sufficient detail, the work is plotted on a 200-ft. scale or a 100-ft. scale, as the case may demand. The record plans are almost invariably made on the 400-ft. scale, in fact, it is a sort of unwritten law that they shall be on that scale. Sometimes (but rarely), for some special purpose, as for a general view of a project for use at a hearing, a map may be made on a smaller scale.

It would seem that there could be no controversy as to the advisability of setting hubs at property lines on location (or on preliminary, if the location is not to follow at once). On location, the writer is a firm believer in the practice of running all tangents to intersection whenever the extra work involved does not utterly prohibit such a course, as will often be the case in rough country.

The writer has made a practice of running all curves of 3° or more with 50-ft. chords, and all sharper than 10° with 25-ft. chords; the intermediate points are always needed when work is cross-sectioned, and time is saved by setting them on location.

In taking level notes all foresights on turning points are kept by themselves, so that there is no chance of confusion when sights are added for checking elevations; and the two things wanted when

Mr. Tripp.

platting a profile—namely, elevations and stations—are in adjoining columns, giving no chance for the rodman to lose his way in getting from one to the other in "calling off." The profile is usually plotted on Plate B paper, but sometimes Plate A is used when the work is light and the grades are not too steep.

The remarks of Mr. Lavis as to the importance of the forechainman are very much to the point; his importance can hardly be over-estimated. It is difficult to get a good man for that position, as it requires a good deal of physical strength and lots of "push," and the combination is hard to find. He should be continually on his feet to keep the axemen on line and cutting to advantage, but should not (as too many are) be continually moving his pole ahead and singing out "line" to the transitman; that over-worked person has troubles enough without adding any needless ones to the list.

As regards the speed of work, of course what is only moderate speed in one place is a rate of progress not to be even thought of in another. The writer was once assistant in a party which, with five axemen, was a long day in getting ahead forty-nine stations on a preliminary line, and the best time that any man in the party made over that distance at night when going to camp was one hour. On other portions of the same survey as many as 4 miles were made during the shortest days.

The writer fully agrees with Mr. Gould in the idea that, after the utmost care has been taken with the projected line, it remains to a considerable extent a cut-and-try process to get the best practicable line on the ground. Usually, when the most carefully projected line has been put on the ground, there will be seen places where a trifling difference in the position of the line will make a difference of many dollars in the cost of construction, and it should always be kept in mind that it takes but a trifling difference in the quantities to pay for a day or two of the time of the locating party. The chief of party should never let pride, on the one hand, or remarks from others (members of the board of directors, for example), on the other, prevent his taking the time for making desirable changes in the line after the location profile has shown them to be advisable. Some men, otherwise thoroughly good locators, cannot grasp the whole of a section and make the best of it, but spend too much time in fitting each separate side hill as closely as possible (apparently) and end by getting far too much curvature in the line, that is, they have curvature beyond the point where it saves enough work to be justifiable. The writer was once Principal Assistant under Frederic Danforth, M. Am. Soc. C. E., and on one portion of the line had the plans of a previous line (if he remembers aright only the preliminary had actually been run—the location being projected). It was on a river bank with a steep side hill, and a very

Mr. Tripp. tortuous line was shown by the plan. In a distance of less than a mile, by judicious treatment of the ground by the Chief Engineer, some 100° of curvature were saved, with no increase of work so far as the profiles could show. No doubt every man of any experience in location can recall similar cases. This is a very interesting subject, and one capable of almost endless discussion from the very fact that what is right in one section is impracticable in another. The writer regrets that others, of those having long experience in the matter, have not found time or inclination to write on the subject.

Mr. Lavis. F. Lavis, Assoc. M. Am. Soc. C. E. (by letter).—While insisting on the superiority of so-called paper location, the writer wishes to emphasize the point that such a projected location should only be made by a man familiar with every detail of the ground, just as familiar as the Geniuses referred to by Mr. Whinery. The writer believes, as in fact was stated in the paper, that the undeniable ability of many of the older locating engineers is just as necessary to-day, but that it should be supplemented by scientific methods and proper surveys. This contention he thinks is fully supported by most of those who have discussed the paper. Where the Genius would go along the hillside fitting the curves to the topography, the plodder would probably be in his tent at night, after he had gone over the ground two or three times, fitting in a tangent where the Genius was fitting the curves, and, ten chances to one, he would accomplish it with the same or less cost for construction.

The methods necessary to attain the desired results must vary with the conditions under which each particular survey is made. In regard to keeping transit notes, the writer believes that the method shown in the paper is generally used, and that it answers all purposes, whether the curve is run forward or backward. At whatever station the instrument is set, or to whatever station it is sighted, the method is always the same: Set the vernier to read the angle belonging to the station sighted at (as a fore-sight) and turn off the angle belonging to the station to be set. For instance, on the curve noted (page 121): Instrument at 735, fore-sight on 738, it being required to set 736; set the vernier at 4° 51′ 14″, sight on 738, turn to 3° 51′ 14″, and set station 736, as required.

The writer prefers to double all angles, instead of reading both verniers, as this is a check on sighting, reading and recording.

In regard to the plane table and stadia, from quite extensive experience of both, and from the opinions of many engineers who have tried these methods on railroad location, and actual data of cost, the writer is convinced that these methods are not applicable to work of this class. He would like to go further into details in this matter, but feels that it would unduly prolong the discussion, and the matter has been already commented upon.

The writer would like to emphasize, particularly, Mr. Whinery's estimate of the topographer. It is difficult to get such men for the salaries generally paid. They, above all things, need an "eye for country;" and that seems to be a gift which is inborn, and not the result of education. Mr. Lavis.

The procedure of the transit party on preliminary lines, as described by Mr. Oakley, is the same as that followed by the writer, and was incorporated with other similar details in the first draft of the paper, but was cut out, as the paper seemed to be quite too long.

The writer would like to correct an apparently mistaken impression of Mr. Howard, that the final location was fixed on the ground wholly by vertical distances. The writer states, page 120: "The points on the located line were fixed by horizontal distances from the hubs on the preliminary, and, also, etc." In ordinary country the horizontal method is preferable, but, where steep slopes are encountered, vertical distances, also, should be used as a check. The writer's experience has been that, with a skilful locating engineer, it is very seldom necessary to back up on the final location, and that in any, except very rough, country, absolutely correct topography is an unnecessary refinement. Good alignment is necessary, and a long tangent is not going to be broken up because a few contours go astray.

Mr. McHenry's definition of engineering, as "The art of making a dollar earn the most interest," is particularly applicable to location. The object of a railroad location survey is, not to get a beautiful topographical map, but to get the best line on the ground for the least cost. Most maps on which the location is projected are made on a scale of 400 ft. to the inch, occasionally, 200 ft. to the inch. At 400 ft. to the inch, it is difficult to plot or see less than 10 ft., hence the absurdity of measuring for contours or using a 5-ft. stick. The height of the eye varies less than a tenth or two, and an engineer who cannot pace 300 ft., within 10 ft., over any kind of ground, ought to go out of the business.

Mr. Howard's methods would increase the cost of getting the topography four or five times, over the methods described in the paper, with no compensating result, if the locating engineer knows his business and is not the machine feared by Mr. Whinery. The real point, which has apparently been missed by those who have discussed the paper, is that too much attention to small details of topography results usually in a line with too much curvature. The engineer's mind is cramped with too much attention to details. Use a small scale, and lay the line down boldly to the general topography, and do not fuss around if some particular little contour goes around the wrong side of a house, or some boulder is not shown. Again, after all the fussing to get the line 5 or 6 in. this way

Mr. Lavis. or that, and fixing the grade line to make the quantities balance exactly, who knows what will happen on construction?

Soundings are taken while locating, to get a general idea of the material, but it often looks very different when the cut is opened, and it often pays to raise the grade line to get over some hard material, or the reverse where no borrow is to be had where expected.

The writer firmly believes that the use of separate sheets is another deterrent to good alignment. The theory may be all right, but in practice it distinctly prevents a study of a long stretch of country. The ordinary drafting table is 5 ft. long, which, at 400 ft. to the inch, gives a range of less than 5 miles. With a roll, the line is worked back and forth, going from one end to the other and back with little trouble. If one has to hunt around and find missing sheets, fumble around for thumb-tacks, and then match the sheets together, the chances are he will let things pass that he would not otherwise, if he had only to unroll his map. It is a good thing, too, although the line on the 2 000-ft. map gives a general view, to run out the roll the length of the tent and get a good look over the whole length of the line, and it will often be found that curves which seem to be all right by themselves are bad when the line is looked at as a whole, and perhaps a little study then will fix them.

The writer, perhaps fortunately for him, has been connected at various times with much poor location, has done not a little himself, and it is because of this and because he thinks he knows most of the ways bad locations are made, that he is perhaps somewhat enthusiastic over the methods which were used on the Choctaw, Oklahoma and Gulf Railroad, and which appealed to him, particularly, as being practical, and as obtaining results, while avoiding all unnecessary refinement of accuracy.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 992.

MAXIMUM RATES OF RAINFALL AT BOSTON.*

By Charles W. Sherman, M. Am. Soc. C. E.

With Discussion by
Messrs. Kenneth Allen, C. E. Gregory, Asa E. Phillips,
E. Kuichling, L. J. Le Conte, William Mayo Venable,
C. S. Burns, S. Whinery, George S. Webster
and Charles W. Sherman.

This paper is in effect a continuation of a paper entitled "Maximum Rates of Rainfall," by Desmond FitzGerald, Past-President, Am. Soc. C. E.,† in which he presented copies of the autographic records of the most important rain storms of high intensity recorded by the rain gauge at Chestnut Hill Reservoir during the ten years, 1879-1888. The objects of the present paper are:

1.—To put on record in tabular form the important data relating to all storms in which the intensity of downpour was considerable, as recorded by the Chestnut Hill gauge from 1879 to 1904, inclusive;

2.—To present copies of the autographic records of rains of especial interest since 1888;

3.—To discuss briefly these records, in comparison with those obtained elsewhere, with especial reference to the relation between the intensity and the duration of the downpour.

* Presented at the Meeting of February 1st, 1905.

† *Transactions*, Am. Soc. C. E., Vol. XXI, p. 93.

TABLE 1.—AMOUNT AND INTENSITY OF "DOWNPOUR" AS SHOWN BY RECORDING RAIN GAUGE AT CHESTNUT HILL RESERVOIR, BOSTON, MASS., 1879–1901.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Total Rainfall, in inches:			Amount of Downpour, in inches:			
Date.	By standard gauge at ground level.	By recording gauge.	Ratio of Column 3 to Column 2	As recorded.	Corrected.	Duration of downpour, in minutes.	Intensity of downpour, in inches per hour.
1879. June 29	1.41	1.07	0.759	0.86	1.13	45	1.51
July 16	....	0.72	.....	0.64	....	15	2.56
Aug. 16-19	6.23	4.56	0.725	3.40	4.68	606	0.46
				0.35	0.48	15	1.98
Sept. 8	0.48	0.44	0.917	0.20	0.22	5	2.62
1880. July 20-21	1.99	1.73	0.869	1.27	1.46	40	2.18
				1.00	1.15	18	3.83
1881. Jan. 10	2.20	2.20	1.000	2.20	2.20	800	0.17
				2.10	2.10	390	0.32
June 10-11	3.83	3.50	0.914	3.50	3.83	2100	0.11
				2.50	2.73	480	0.34
July 21	0.48	0.36	0.750	0.35	0.47	12	2.33
Sept. 2-3	1.79	1.74	0.972	0.76	0.78	35	1.34
1882. July 19	0.32	0.32	1.000	0.32	0.32	13	1.48
" "	0.48	0.43	0.896	0.40	0.45	18	1.49
Sept. 14	1.13	1.05	0.929	1 00	1.08	45	1.44
Sept. 22-23	2.55	2.38	0.934	2.38	2.55	765	0.20
				1.50	1.61	165	0.58
1883. June 29	0.59	0.56	0.950	0.56	0.59	30	1.18
				0.40	0.42	8	3.16
1884. June 19	1.66	1.55	0.934	1.34	1.43	68	1.26
				1.55	1.66	240	0.42
July 18	1.19	1.14	0.958	0.67	0.70	5	8.40
				1.14	1.19	135	0.53
1885. June 28-29	1.26	1.15	0.913	0.97	1.06	98	0.65
				1.15	1.26	470	0.16
June 29	1.17	1.05	0.897	0.93	1.04	68	0.92
				1.05	1.17	215	0.33
				0.60	0.67	15	2.68
July 29	0.81	0.76	0.938	0.76	0.81	125	0.39
				0.50	0.53	25	1.28
Aug. 1	1.76	1.61	0.915	1.17	1.28	70	1.10
				1.61	1.76	365	0.29
				0.25	0.27	1	16.40*
1885. Oct. 2-3	1.25	1.18	0.945	0.30	0.32	7	2.72
1886. July 15	1.74	1.59	0.914	1.59	1.74	450	.23
				0.30	0.33	8	2.46
				0.36	0.39	10	2.36
				0.37	0.40	15	1.62
1887. July 10	1.10	0.99	0.900	0.41	0.46	15	1.82
Aug. 20	0.33	0.30	0.909	0.30	0.33	8	2.48
Oct. 1	1.00	0.96	0.960	0.35	0.37	20	1.09
1888. May 13	1.32	1.22	0.924	1.22	1.32	420	0.19
				1.20	1.30	150	0.52
Aug. 12-13	1.73	1.53	0.844	1.04	1.18	60	1.18
				0.96	1.09	45	1.45
Aug. 21-22	3.44	3.23	0.939	3.02	3.22	200	0.97
				3.23	3.44	540	0.38
				1.50	1.60	60	1.60
				2.43	2.58	120	1.29
				3.06	3.26	180	1.09
Sept. 10	1.35	1.30	0.963	1.24	1.29	190	0.41
Sept. 17-18	1.94	1.78	0.918	1.17	1.37	120	0.64
				0.80	0.87	60	0.87

* Time interval estimated on diagram as not more than 1 minute. It is possible that float had been stuck and was released at this point, but diagram shows nothing to cast discredit on the record.

TABLE 1—(*Continued*).

(1) Date.	(2) Total Rainfall, in inches: By standard gauge at ground level.	(3) Total Rainfall, in inches: By recording gauge.	(4) Ratio of Column 3 to Column 2	(5) Amount of Downpour, in inches: As recorded.	(6) Amount of Downpour, in inches: Corrected.	(7) Duration of downpour, in minutes.	(8) Intensity of downpour, in inches per hour.
1889. May 20-21....	3.17	3.19	1.006	1.30	1.29	105	0.74
				0.55	0.55	25	1.31
				0.15	0.15	6	1.50
June 2........	1.59	1.55	0.975	0.30	0 31	9	2.05
June 5........	0.47	0.47	1.000	0.30	0.30	6	3.00
June 17.......	0.45	0.44	0.978	0.30	0.31	15	1.23
July 17.......	0.75	0.68	0.906	0.47	0.52	11	2.82
Aug. 1........	0.51	0.48	0.941	0.25	0.27	4	3.98
Aug. 14.......	1.60	1.51	0.944	0.31	0.33	12	1.64
Sept. 11......	1.35	1.04	0.770	0.20	0.26	5	3.12
1890. July 26.......	0.96	0.91	0.948	0.91	0.96	225	0.26
				0.83	0.88	60	0.88
				0.20	0.21	3	4.21
July 31.......	0.47	0.44	0.936	0.42	0.45	30	0.90
Aug. 6........	0.31	0.34	1.096	0.30	0.27	5	3.28
Aug. 19-20,...	0.75	0.72	0.960	0.40	0.42	11	2.27
1891. Sept. 5-7......	2.83	2.55	0.902	1.25	1.38	60	1.38
				0.50	0.55	15	2.22
Oct. 7-8	2.45	2.12	0.866	2.12	2.45	760	0.19
				1.90	2.19	270	0.49
				0.60	0.69	45	0.92
1892. June 17.......	1.18	1.07	0.907	1.07	1.18	110	0.64
				0.80	0.88	30	1.76
				0.52	0.57	15	2.29
July 3........	0.74	0.66	0.892	0.27	0.30	7	2.59
1892. July 25.......	0.44	0.34	0.772	0.32	0.41	15	1.65
Aug. 12.......	2.19	2.17	0.990	2.17	2.19	500	0.26
				1.30	1.31	30	2.62
				0.29	0.29	11	1.60
1893. May 27........	0.48	0.49	1.021	0.40	0.39	17	1.38
July 18.......	0.73	0.73	1.000	0.73	0.73	45	0.97
July 22.......	0.67	0.74	1.105	0.30	0.27	12	1.35
Aug. 6-7......	1.48	0.95	0.642	0.52	0.81	11	4.41
1894. July 21-22....	1.44	1.43	0.993	1.07	1.08	22	2.94
July 25.......	0.82	0.77	0.939	0.25	0.27	3	5.32
Aug. 20.......	1.40	1.42	1.014	1.40	1.38	280	0.30
				0.60	0.59	13	2.73
				1.00	0.99	35	1.69
Sept. 8........	0.34	0.34	1.000	0.24	0.27	7	2.12
1895. July 13.......	0.45	0.45	1.000	0.23	0.23	7	1.97
July 30.......	0.63	0.63	1.000	0.60	0.60	90	0.40
Aug. 7........	1.48	1.45	0.980	1.45	1.48	265	0.34
				0.35	0.36	8	2.68
Aug. 18.......	1.17	1.20	1.026	1.20	1.17	285	0 25
				0.80	0.78	30	1.56
				0.40	0.39	7	3.34
Sept. 11.......	0.75	0.75	1.000	0.70	0.70	17	2.47
Oct. 12-14....	7.55	....	....	....	....	2 325	0.19
Oct. 31-Nov. 1	2.86	2.43	0.850	1.50	1.76	180	0.59
Nov. 14-15.....	1.86	1.60	0.860	0.40	0.47	25	1.12
				0.25	0.29	9	1.94
1896. Sept. 5-6......	1.67	1.63	0.976	0.20	0.21	6	2.05
1897. June 15.......	0.32	0.32	1.000	0.30	0.30	9	2.00
July 28.......	1.74	1.57	0.902	0.88	0.96	50	1.17
				0.72	0.80	28	1.71
				0.62	0.69	20	2.06
				0.40	0.44	10	2.66
Aug. 4........	0.38	0.35	0.920	0.30	0.33	15	1.20
				0.16	0.17	4	2.61

TABLE 1—(*Continued*).

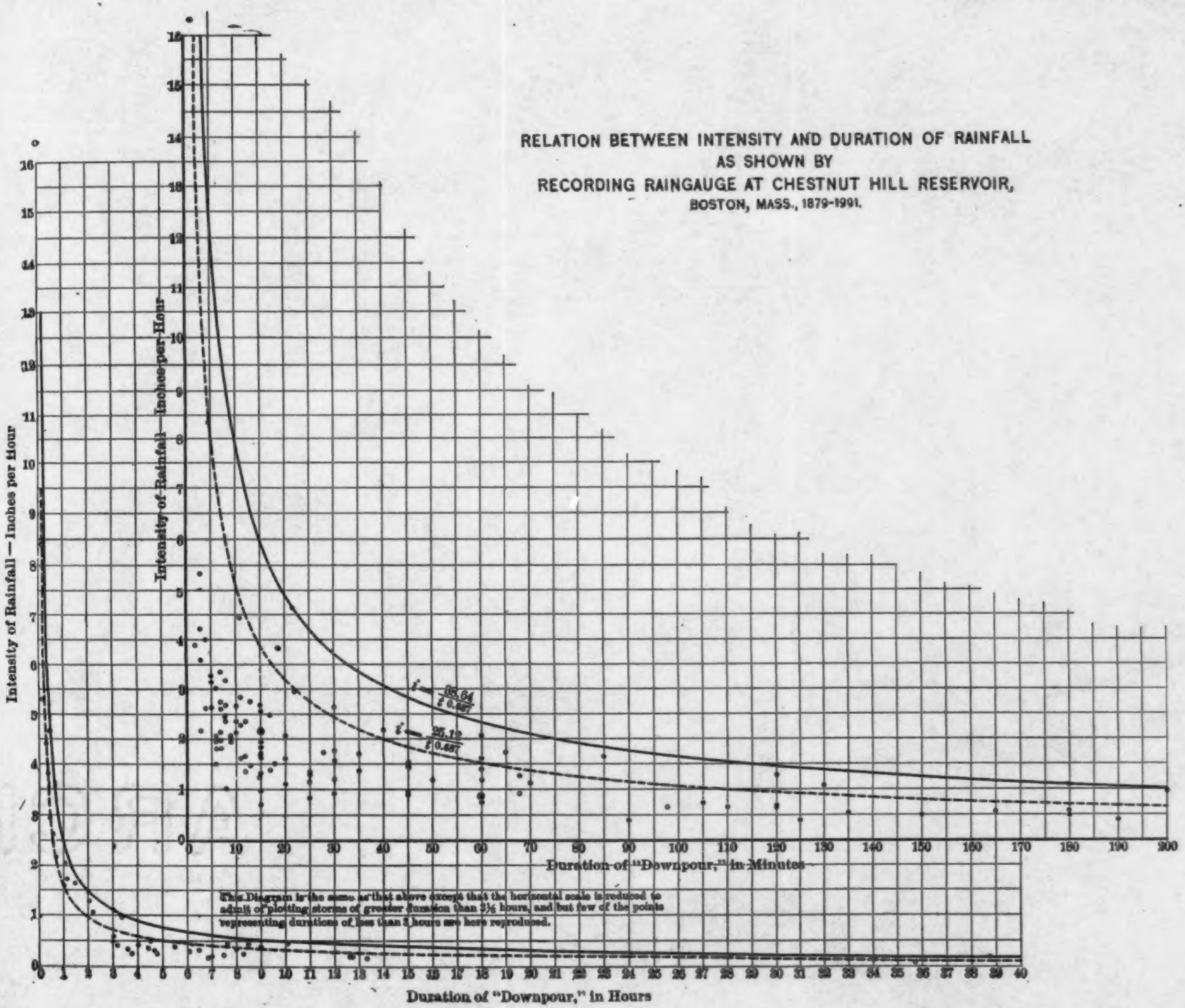
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Total Rainfall, in inches:			Amount of Downpour, in inches:			
Date.	By standard gauge at ground level.	By recording gauge.	Ratio of Column 3 to Column 2	As recorded.	Corrected.	Duration of downpour, in minutes.	Intensity of downpour, in inches per hour.
1897. Aug. 22	0.62	0.66	1.064	0.57	0.54	20	1.61
Sept. 16	0.33	0.33	1.000	0.18	0.18	6	1.80
Sept. 20	0.98	1.00	1.021	0.56	0.55	15	2.19
1898. July 4	0.75	0.81	1.080	0.68	0.63	30	1.26
				0.59	0.55	15	2.19
Oct. 21-22	1.49	1.25	0.839	0.15	0.18	3	3.57
1899. July 8	0.59	0.50	0.848	0.11	0.13	2	3.90
July 26	0.89	0.82	0.921	0.11	0.12	6	1.95
July 27	0.29	0.29	1.000	0.21	0.21	7	1.80
Aug. 22	2.33	2.27	0.974	2.27	2.33	85	1.65
				1.65	1.69	22	4.62
				0.17	0.17	15	0.70
				0.41	0.42	15	1.68
				2.00	2.05	60	2.05
Sept. 20	3.34	3.18	0.952	3.18	3.34	460	0.44
				2.20	2.31	130	1.07
Oct. 18	0.61	0.54	0.885	0.30	0.34	25	0.81
1900. Jan. 25	0.57	0.55	0.965	0.13	0.13	8	1.01
Aug. 15	0.39	0.40	1.023	0.25	0.24	15	0.98
Aug. 16	0.72	0.67	0.930	0.10	0.11	3	2.15
Sept. 17	2.02	1.80	0.891	1.30	1.46	180	0.49
				1.10	1.23	120	0.62
Nov. 9	0.49	0.47	0.959	0.30	0.31	8	2.35
1901. July 29	2.10	1.90	0.905	1.90	2.10	330	0.38
				0.70	0.77	60	0.77
Aug. 7	1.45	1.43	0.986	0.35	0.35	10	2.13
Aug. 25	2.10	2.20	1.047	1.95	1.86	65	1.72
1904. Sept. 14-15	3.84	3.78	0.984	3.56	3.62	510	0.43

1.—Data of Maximum Rates of Rainfall at Chestnut Hill Reservoir, Boston, 1879-1904.

The Chestnut Hill recording rain gauge was established in 1878-79 by Desmond FitzGerald, then Superintendent of the Western Division of the Boston Water-Works, and was maintained under his direction by the Boston and Metropolitan Water-Works until his resignation from that service in 1902; since then it has been in charge of Dexter Brackett, M. Am. Soc. C. E., to whom, as well as to Frederic P. Stearns, M. Am. Soc. C. E., Chief Engineer of the Metropolitan Water-Works, the writer is indebted for permission to publish these records.

Table 1 contains the important data, except that very few storms having a duration of more than 10 hours are considered.

RELATION BETWEEN INTENSITY AND DURATION OF RAINFALL
AS SHOWN BY
RECORDING RAINGAUGE AT CHESTNUT HILL RESERVOIR,
BOSTON, MASS., 1879-1901.



2.—Copies of Records.

The character of the records and the usual characteristics of the rain storms of high intensity are shown so well by the plates accompanying Mr. FitzGerald's paper that no useful object would be served by the presentation of other records, except in the case of storms of more than ordinary interest. Fig. 1 contains the records of three such storms.

The record for July 21st and 22d, 1894, showed a downpour amounting to 1.08 in. in 22 min., when corrected for elevation of the gauge, or a rate of 2.94 in. per hour; and this rate was maintained with almost absolute uniformity for the whole 22 min.

COPY OF RECORDS MADE BY RECORDING RAIN GAUGE.
OF CHESTNUT HILL RESERVOIR, BOSTON, MASS.

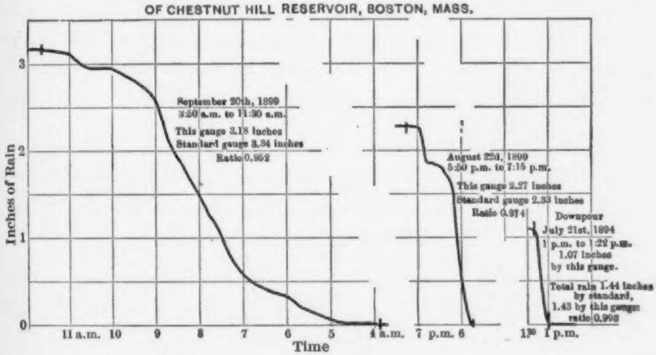


Fig. 1.

The rain of August 22d, 1899, is one of the most interesting ever recorded by this gauge, in that it shows a precipitation in excess of 2 in. in a single hour (to be precise, 2.05 in. in 60 min.). It also shows the unusual intensities of 1.65 in. per hour for 85 min., and 4.62 in. per hour for 22 min. As far as the writer is aware, but one other storm having a greater intensity than 2 in. per hour for 60 min. has ever been registered by a recording rain gauge in the eastern part of the United States, that of August 3d, 1898, at Philadelphia.*

* This most interesting storm, reported by Mr. A. J. Henry, of the U. S. Weather Bureau, in the *Journal* of the Western Society of Engineers for April, 1899, had intensities as follows:

7.20	in.	per	hour	for	5	min.
5.12	"	"	"	"	15	"
4.02	"	"	"	"	40	"
3.78	"	"	"	"	70	"
2.96	"	"	"	"	110	"

The rain of September 20th, 1899, is of interest principally on account of the long time during which a comparatively high intensity was maintained, 1.07 in. per hour for 2 hr. 10 min.

3.—Discussion and Comparison of Records.

The relation between the intensity of precipitation and the duration of downpour for each of the storms included in Table 1 is shown on the diagrams, Plate XV. The upper or full curve is intended to represent the maximum intensity of rainfall for any period, as far as it may be determined from these records. Its equation is $i = \frac{38.64}{t^{0.687}}$, in which i represents intensity of precipitation, in inches per hour; and t its duration, in minutes. It will be noted that none of the observed points falls beyond this curve. The lower or broken curve is intended to represent the greatest intensity of precipitation for any period which it would ordinarily be necessary to consider in engineering design, storms of greater intensity being of rare occurrence. It is represented by the equation, $i = \frac{25.12}{t^{0.687}}$.

E. S. Dorr, M. Am. Soc. C. E., studying the records of this rain gauge, when the series included only 14 years, concluded that the expression, $i = \frac{150}{t + 30}$, included all rainfalls which it would be necessary to consider in designing combined sewers for the City of Boston.* For periods greater than 20 min., this curve differs but slightly from the lower curve proposed in Plate XV, as may be seen by reference to Plate XVI.

Comparison with other Records.—A. N. Talbot, M. Am. Soc. C. E., has noted that the maximum rates of rainfall seem to be very uniform in different parts of the United States (east of the Rocky Mountains), and has deduced a maximum and an ordinary curve as applicable to the whole country.† It is of interest, therefore, to compare with these records and curves those obtained from other observations. Such a comparison is shown on Plate XVI, on which are plotted the following lines:

A. "Maximum" curve for Boston, proposed in this paper;

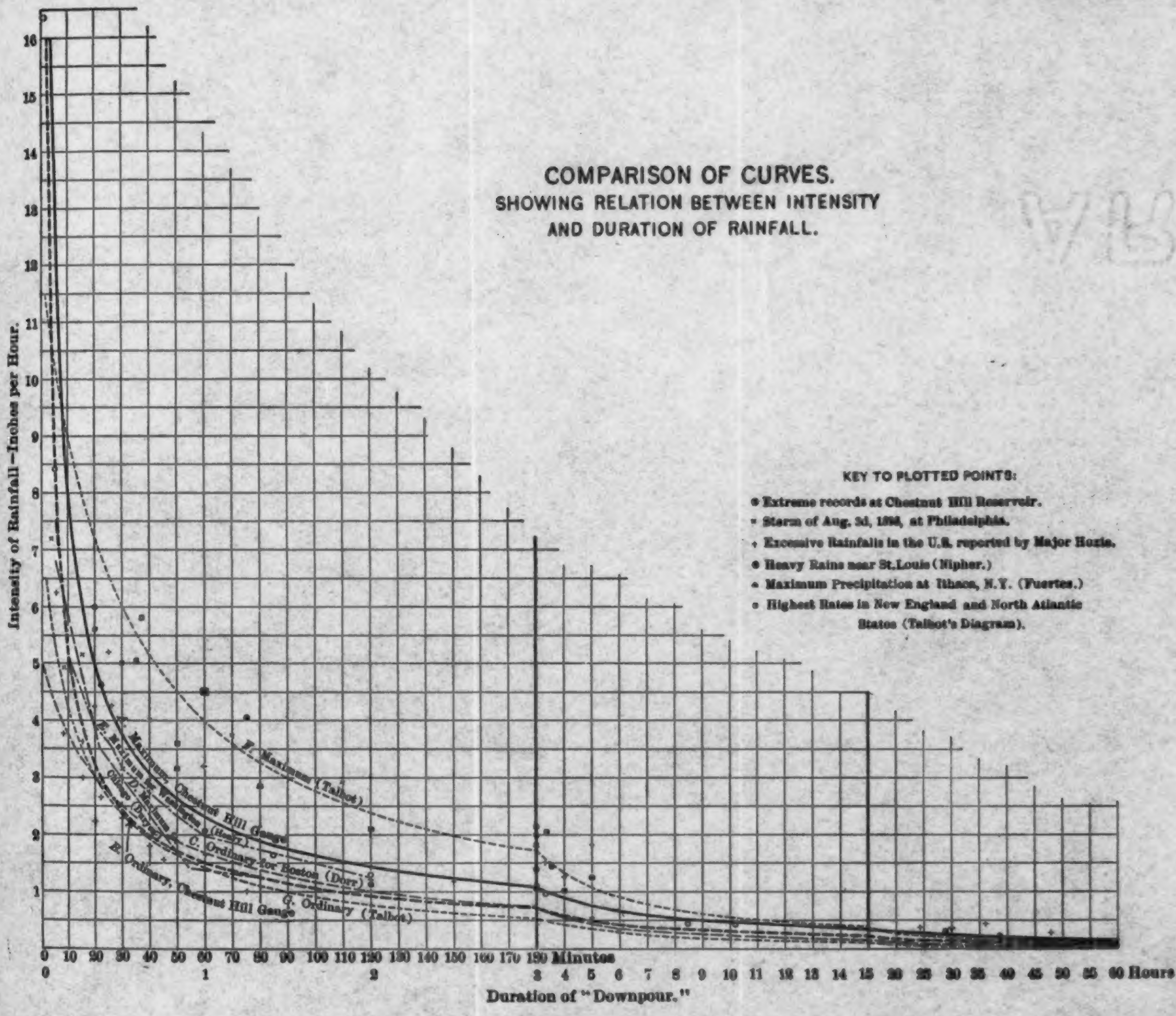
$$i = \frac{38.64}{t^{0.687}}.$$

* "Report of Street Department, Boston," 1892, p. 117.

† "Rates of Maximum Rainfall," *The Technograph*, 1891-92. Abstracted in *Engineering News*, July 21st, 1892, p. 67.

PLATE XVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 992.
SHERMAN ON
MAXIMUM RATES OF RAINFALL.

COMPARISON OF CURVES.
SHOWING RELATION BETWEEN INTENSITY AND DURATION OF RAINFALL.



B. "Ordinary" curve for Boston, proposed in this paper;

$$i = \frac{25.12}{t^{0.687}}.$$

C. "Ordinary" curve for Boston, proposed by E. S. Dorr in 1892;

$$i = \frac{150}{t + 30}.$$

D. Line of maximum rainfalls at Chicago, as determined by Edwin Duryea, Jr., M. Am. Soc. C. E.*

E. Line of maximum intensity of rainfall at Washington for the 16 years, 1881–1896, as reported by Mr. Alfred J. Henry.†

F. "Maximum" curve for eastern United States, proposed by A. N. Talbot, M. Am. Soc. C. E.;‡

$$i = \frac{360}{t + 30}.$$

G. "Ordinary" curve proposed by Professor Talbot;

$$i = \frac{105}{t + 15}.$$

In addition to these curves, points have been plotted on Plate XVI to show excessive rates of rainfall noted at other stations in the United States, as well as the extreme rates recorded at Chestnut Hill Reservoir. These points are:

a. Extreme records at Chestnut Hill Reservoir. (Marked by circles.)

b. Storm of August 3d, 1898, at Philadelphia.§ (Marked by crosses.)

c. All the excessive rainfalls reported by Major R. L. Hoxie, M. Am. Soc. C. E., which occurred in the United States.‖ This includes the great rain of October 3d and 4th, 1869, reported by the late James B. Francis, Past-President, Am. Soc. C. E.;¶ also two storms reported in Trautwine's "Civil Engineer's Pocket Book" as follows:

"In July, 1842, 6 inches fell in 2 hours. . . . During a tremendous rain at Norristown, Pa., in 1865, the writer saw

---

* "Tables of Excessive Precipitations of Rain at Chicago, Ill., from 1889 to 1897, Inclusive." *Journal* of the Western Society of Engineers, February, 1899.

† "Excessive Precipitation in the United States." *Monthly Weather Review*, January, 1897. (Abstract in *Engineering News*, June 24th, 1897, p. 386.)

‡ *The Technograph*, 1891–1892; *Engineering News*, July 21st, 1892.

§ A. J. Henry, in *Journal* of the Western Society of Engineers, April, 1899.

‖ "Excessive Rainfalls Considered with Especial Reference to their Occurrence in Populous Districts." *Transactions*, Am. Soc. C. E., Vol. XXV, p. 70.

¶ *Transactions*, Am. Soc. C. E., Vol. VII, p. 224.

evidence that at least 9 inches fell in 5 hours." (Points marked by plusses.)

d. Heavy rains near St. Louis, reported by Francis E. Nipher in a letter to the *American Engineer*, May 8th, 1885. (Marked by crosses within circles.)

e. The maximum precipitation registered by the recording rain gauge at Ithaca, N. Y., as reported by James H. Fuertes, M. Am. Soc. C. E.* (Marked by a triangle.)

f. The highest points shown on Professor Talbot's diagram for the New England and North Atlantic States, in the paper previously quoted. (Marked by rectangles.)

In examining Plate XVI it is at once seen that, with the exception of Professor Talbot's "maximum" curve, the "maximum" curve obtained from the Chestnut Hill records is the highest of any; and that the "ordinary" curve derived in this paper does not differ widely from Professor Talbot's "ordinary" curve, nor from any of the others shown.

With reference to the points lying above the Chestnut Hill "maximum" curve, it should be remembered that the only ones obtained by a recording rain gauge are those representing the Philadelphia storm of August, 1898. All the others, therefore, are open to question to a greater or less degree. The Philadelphia record, however, at once lends credence to the others as being at least approximately correct. It appears, then, that Professor Talbot's "maximum" curve must probably be accepted as representing an intensity of rainfall that should be expected perhaps once in a century; except that this curve seems to give results which are too small for durations of less than 5 min. or more than about 15 hours. It appears, also, that, although there is nothing in the Chestnut Hill records to support such a conclusion, such extreme rainfalls must be expected at Boston, since there have been a number of sufficiently well authenticated storms of this character at no great distance from Boston, as well as the autographically recorded storm at Philadelphia in 1898.

Finally, it would appear that, for ordinary engineering design, such a rainfall as would be shown by either of the three "ordinary" curves—Mr. Dorr's, Professor Talbot's, or that proposed in this paper

* "Rates of Precipitation in Rain Storms at Ithaca, N. Y." *Engineering News*, September 20th, 1894, p. 226.

—would be as heavy as there would usually be necessity for considering. The writer believes the last curve to be better supported by the Chestnut Hill records. In extreme cases it may be necessary to consider rainfalls as heavy as those shown by the "maximum" curve herein proposed. Such rainfalls may perhaps be expected as often as once in 8 or 10 years. Finally, such extreme rainfalls as would be shown by Professor Talbot's "maximum" curve (between the limits of 5 min. and 15 hours, beyond which this curve gives results which are too low) must be expected to occur at long intervals, perhaps about once in a century.

DISCUSSION.

KENNETH ALLEN, M. AM. SOC. C. E. (by letter).—Previous to 1880 there was very little reliable information concerning the intensity of rainfall during short periods. Since then, by the introduction of self-registering gauges, many data, particularly valuable to the engineer in the design of municipal work, have been secured, and the marked similarity between the several curves shown by the author, with the exception of Talbot's Maximum, is a measure of the reliability of the several series of records upon which they were based. Mr. Allen.

The writer has kept rough memoranda of excessive rainfalls for a good many years, and, in looking them over, finds the following

Mr. Allen. (not already mentioned by the author) that lie above the Maximum Chestnut Hill curve:

Of the thirty-one rates of precipitation noted, eighteen of which are for the Eastern States, seventeen exceed the rates for corresponding intervals shown by Professor Talbot's Maximum curve, and, of the latter, but six are for periods of less than 2 hours. Rates for shorter periods almost invariably fall within the Maximum Chestnut Hill curve.

TABLE 2.—RATES OF PRECIPITATION.

Rates of Precipitation Lying Between the Maximum Chestnut Hill and Talbot Curves.

Locality.	Date.	Rate of Precipitation, in inches.	Duration. Hours.	Duration. Minutes.
Kansas City, Mo	June 17, 1895.	5.25	0	20
Kansas City, Mo	May 12, 1886.	6.00	0	25
Indianapolis, Ind.	July .., 1876.	5.76	0	25
Washington, D. C	July .., 1881.	3.78	0	37
Chelsea, Vt.	July 6, 1897.	3.68	1	00
Newark, N. J.	Aug. 24, 1897.	2.95	1	00
St. Louis, Mo.	July 8, 1898.	2.48	1	02
Baltimore, Md.	July 11, 1884.	2.18	1	43
Norfolk, Va	Aug. 14, 1898.	2.54	1	52
Jacksonville, Fla.	June .., 1903.	1.68	2	30
Atlantic City, N. J	June 22, 1903.	1.94	2	30
South Canistea, N. Y.	May 31, June 1, 1889.	1.50	3	00
Reading, Pa.	Oct. 12, 1897.	1.36	3	00
Atlantic City, N. J	June 22, 1903.	1.01	4	30
Rates of Precipitation Lying Above the Maximum Talbot Curve.				
Galveston, Tex.	June .., 1871.	16.9	0	14
Embarras, Wis.	May .., 1881.	9.2	0	15
Sandusky, Ohio.	July .., 1879.	9.0	0	15
Brattleboro, Vt.	July 7, 1897.	6.78	0	20
Biscayne, Fla.	March .., 1874.	8.2	0	30
Newton, Pa.	Aug. .., 1843.	8.25	0	40
Brandywine, Pa.	Aug. 5, 1843,	5.00	2	00
Newark, N. J.	Aug. 24, 1897.	2.58	2	20
Concordia, Pa.	Aug. 5, 1843.	5.33	3	00
Newtown, Del.	Aug. 5, 1843.	4.33	3	00
2 miles above Canistea, N. Y.	May 31, June 1, 1889.	2.00	3	00
Wellsboro, N. Y.	May 31, June 1, 1889.	0.75	10	00
Cuyamoca Dam, San Diego Co., Cal.	Feb. .., 1891.	0.70	10	00
Meriden, Miss.	Apr. 15, 16, 1900.	0.77	12	00
Jewel, Md.	July 26, 7, 1897.	0.82	18	00
Mayport, Fla.	Sept. 29, 1882.	0.57	24	00
Cuyamoca Dam, San Diego Co., Cal.	Feb. .., 1891.	0.43	54	00

Although more data are desirable, in order to form conclusions, the writer, from the fact that his complete list includes a larger pro-

portion of observations for short periods, is inclined to think that, for periods of 2 hours or more, the curves shown for maximum rates of precipitation are relatively low; and that a curve, like Talbot's Maximum, that barely excludes a small number of phenomenal downpours for short periods, might well be extended so as to include a rate of ½ in. for 24-hour periods. Mr. Allen.

Although it is not known whether or not the gauges were self-registering, it will be noted that there are three instances in Table 2 —at Newark, N. J., Chelsea, Vt., and St. Louis, Mo.—in which a rate of more than 2 in. was maintained for an hour.

C. E. GREGORY, ASSOC. M. AM. SOC. C. E.—In comparing the valuable data given in this paper with data already at hand, it occurred to the speaker that, by using one of the expressions for intensity, a more rational and general formula than those in general use might be deduced to show the relation of rainfall to run-off; one that would not only simplify this problem of the relation of run-off to rainfall, but also take into account the shape of the water-shed and possibly serve as a basis for harmonizing, somewhat, the conflicting data. Mr. Gregory.

First assume that, for all water-sheds of equal area and of similar shape, character and slope, there should be a fairly constant ratio between rainfall and run-off for storms of equal duration and intensity. In practice, however, all water-sheds and all storms are different. Therefore, each different storm and water-shed must be reduced to a common basis of comparison. The maximum flow from all water-sheds, except those of unusual shape, obviously, is at the time when all parts of the area are first contributing their average maximum to the outlet at the same time. For any shorter period only part of the area would be contributing, and, for a longer period, the intensity would be less.

The qualifying factor, in any water-shed, which cannot be expressed by a direct coefficient, is the time it takes the rain, falling on the roofs and streets in the most distant part of the water-shed, to reach the point under consideration.

The qualifying factors in the storm are its duration and intensity.

All storms, however, may be reduced to a basis of comparison on a particular water-shed by comparing the greatest average intensity for the time the water takes to travel to the point in question, rather than the maximum intensity for a shorter time, or the average for the entire storm. Storms compared on this basis show a remarkably constant relation to the maximum run-off, while, if compared in other ways, they show great variation. Unfortunately, the speaker, as yet, has been able to obtain but very few data which are in proper form to make this comparison, and has been confined, almost

Mr. Gregory. entirely, to data taken from a report by Rudolph Hering, M. Am. Soc. C. E., on gaugings taken in 1888 in the Sixth Avenue Sewer, in New York City. These have shown a ratio of from 50 to 60% of the average intensity for the proper time for the maximum rate of run-off.

The relation of intensity of storms to their duration is most simply expressed by Professor Talbot's "Ordinary" form of curve, $i = \frac{105}{t + 15}$.

To find the proper intensity for any water-shed, substitute for t, $\frac{l}{v}$, in which l is the greatest length of the water-shed, and v is the average velocity, in feet per minute; then, calling inches per hour equivalent to cubic feet per second per acre, $Q = A\ C\ i$, in which Q = cubic feet per second, A = acres, and C is the coefficient of the relation of run-off to rainfall, and will vary with the nature of the surface and the drainage system; then, for i, substitute its value and allow 10 minutes for the water to reach the sewer from the roofs and streets, and the result is

$$Q = A\ C \frac{105}{\frac{l}{v} + 15 + 10} \qquad \text{(1)}$$

To make the formula general, express v in terms of A and S, S being the average slope of the water-shed, in feet per thousand. To do this it is necessary to substitute in the standard formulas for velocity, $v = c\sqrt{R\ S}$, and $Q = a\ v$; in which c is a variable coefficient; R is the hydraulic mean radius; S is the sine of the slope of the water surface; v is the velocity, in feet per second; a is the area of the sewer section, and Q is the discharge, in cubic feet per second. Giving to c an average value of 120, and expressing S in feet per thousand, the result is

$$v = 120\sqrt{R \times \frac{S}{1\ 000}}$$

$$\text{and } v^2 = 14.400\ R\ S,$$

$$\text{or } v = 3.8\sqrt{R\ S}.$$

Also, $Q = a\ v$, and $v = \frac{Q}{a}$.

If a = area = $\frac{\pi\ d^2}{4}$, and $R = \frac{d}{4}$, assign an average value for $\sqrt{R}$, say $\frac{1}{2}\sqrt{R}$ or $\sqrt{\frac{R}{4}}$; then the average value, expressed in terms of d, is $\frac{d}{16}$, or $d = 16\ R$. Substitute for d its value, and

$a = 192\,R^2$; then substitute for a its value, and $v = \dfrac{Q}{192\,R^2}$, Mr. Gregory.

$$\text{but } v = 3.8 \sqrt{R\,S},$$

therefore $3.8 \sqrt{R\,S} = \dfrac{Q}{122\,R^2}$,

and $$R^{\frac{5}{2}} = \frac{Q}{730 \sqrt{S}},$$

or $$R^{\frac{1}{2}} = \sqrt[5]{\frac{Q}{730 \sqrt{S}}}$$

$$= \frac{1}{3.74} \sqrt[5]{\frac{Q}{\sqrt{S}}}.$$

Substitute for $\sqrt{R}$ its value, and

$$v = \frac{3.8}{3.74} \sqrt[5]{\frac{Q}{\sqrt{S}}} \times \sqrt{S}$$

$$= \sqrt[5]{Q\,S^2},$$

very nearly.

Then, as A is an average value for Q, it is sufficiently accurate to substitute it for Q, and express v in cubic feet per minute, or sixty times the value above, thus

$$Q = A\,C \frac{105}{\dfrac{l}{60 \sqrt[5]{A\,S^2}} + 25} \quad \ldots\ldots\ldots\ldots (2)$$

as the general form for the relation of run-off to rainfall, and this can be compared directly with any rainfall data which may be desirable.

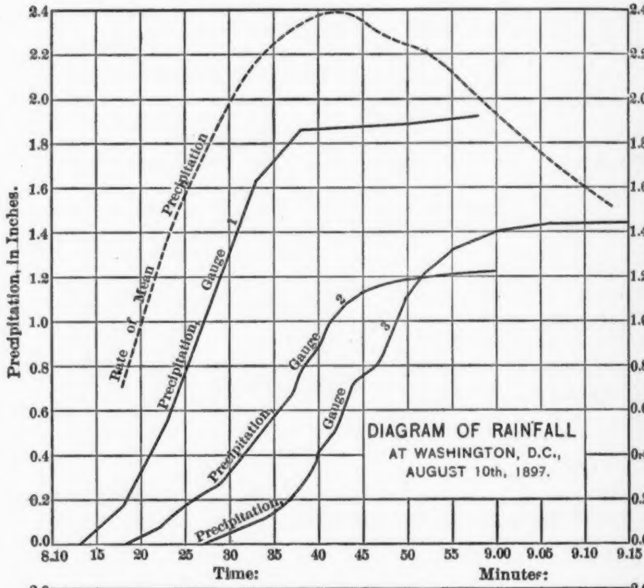
This formula gives for average-shaped water-sheds values somewhat similar to McMath's when 0.80 is used for C and 2.75 for r, but varies more nearly with the slope and gives smaller results for very long narrow water-sheds and larger ones for those which are fan-shaped.

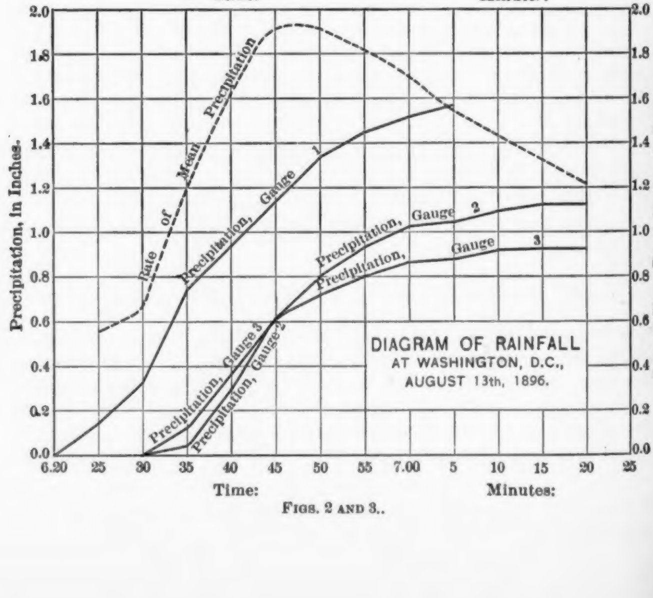
If great accuracy is desired, the value of the term, $\dfrac{l}{\sqrt[5]{A\,S^2}}$, may be tested, after the sizes and velocities have been worked out, by taking the actual velocities and using Formula 1, and, if desirable, the time allowed for the rain to reach the sewers may be changed.

It should be noted that the area tributary to the Sixth Avenue Sewer gauged by Mr. Hering consisted of 43.5% roof area, 46.5% paved area and 10% grass area.

Asa E. Phillips, M. Am. Soc. C. E. (by letter).—The following is submitted as a local contribution to the general subject of maximum rates of rainfall: Mr. Phillips.

Mr. Philips.





FIGS. 2 AND 3..

Mr. Phillips.

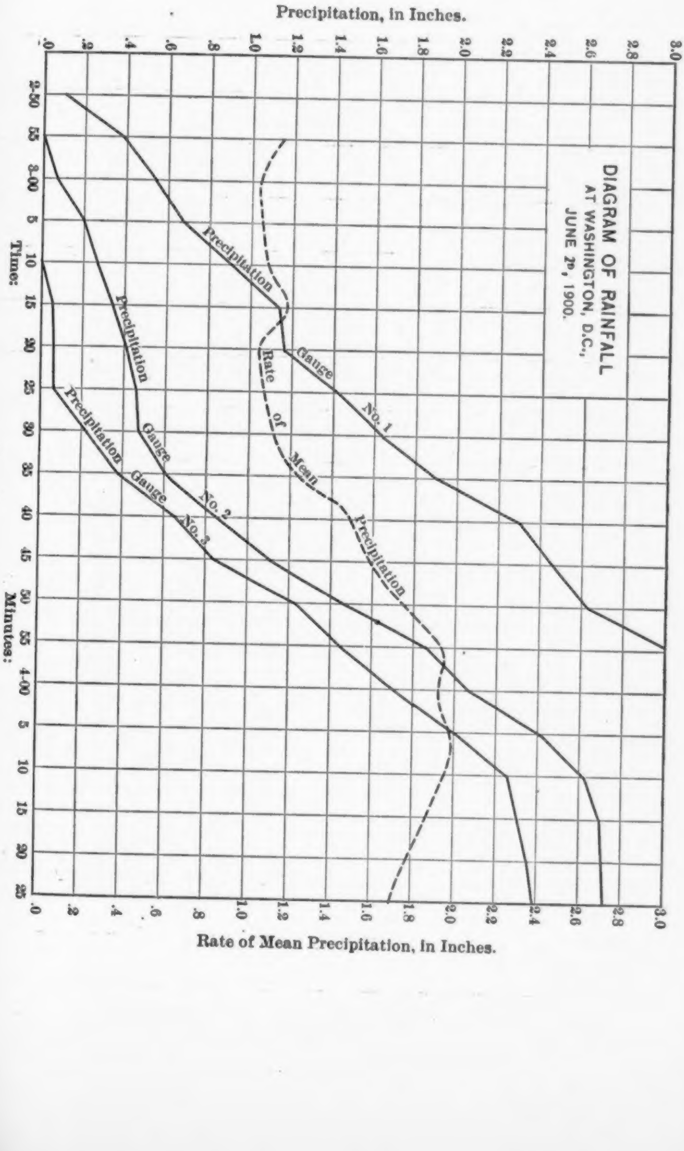


FIG. 4.

Mr. Phillips.

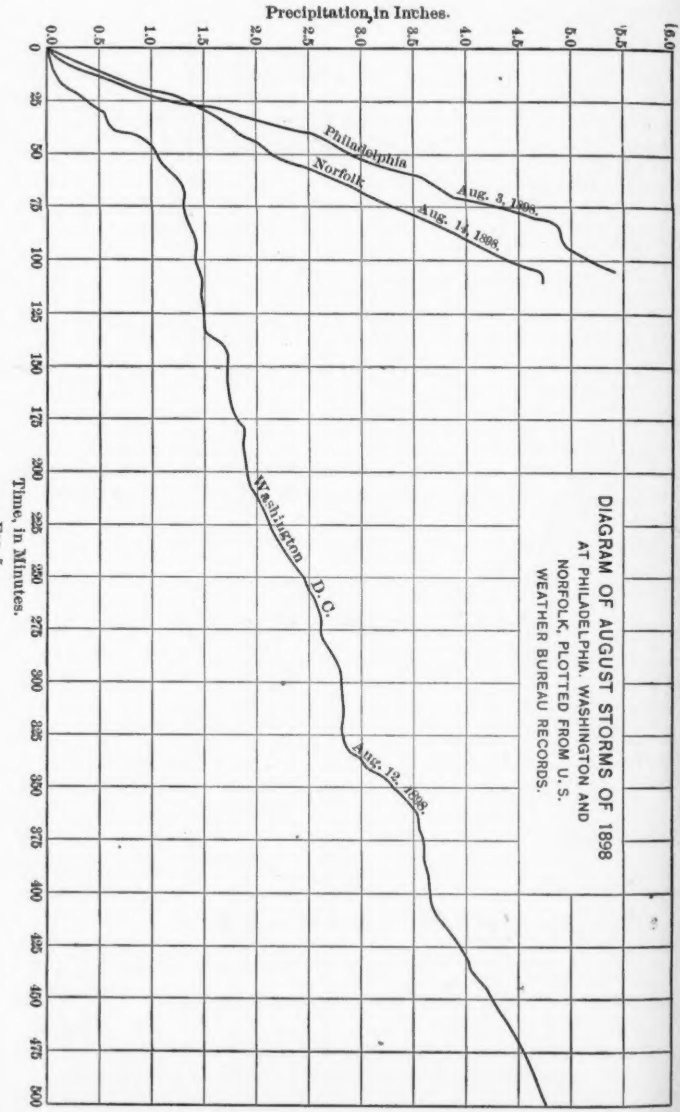


Fig. 5.

Mr. Phillips.

TIME INTERVAL BETWEEN MAXIMUM RATES OF PRECIPITATION
AS RECORDED AT EACH STATION, FOR EXCESSIVE STORMS,
AT WASHINGTON, D.C., 1890 TO 1904.

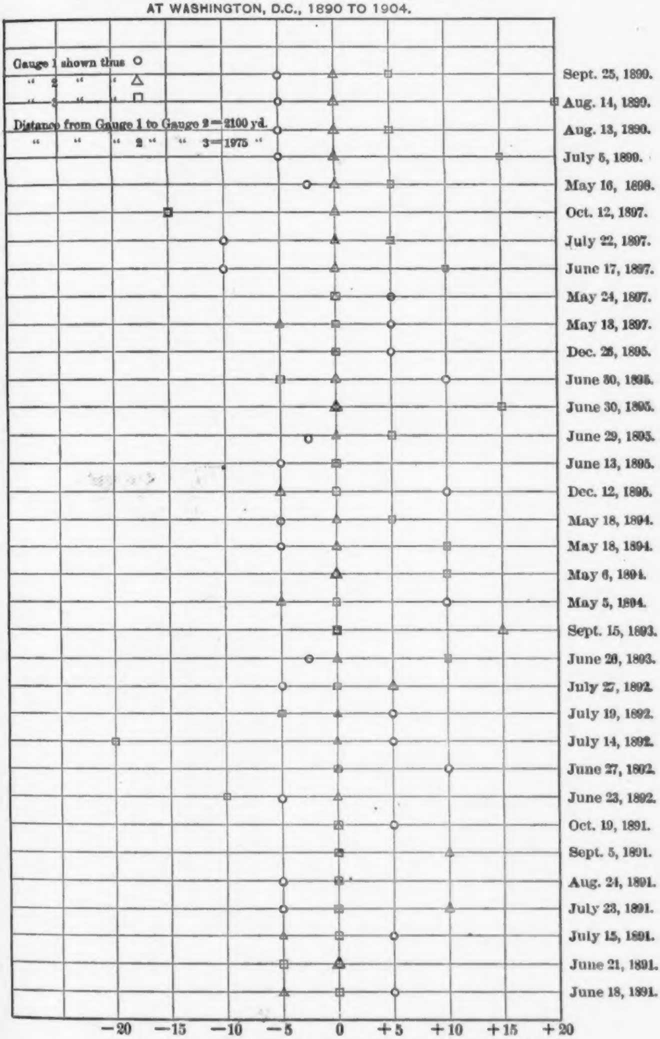


FIG. 6.

Mr. Phillips.

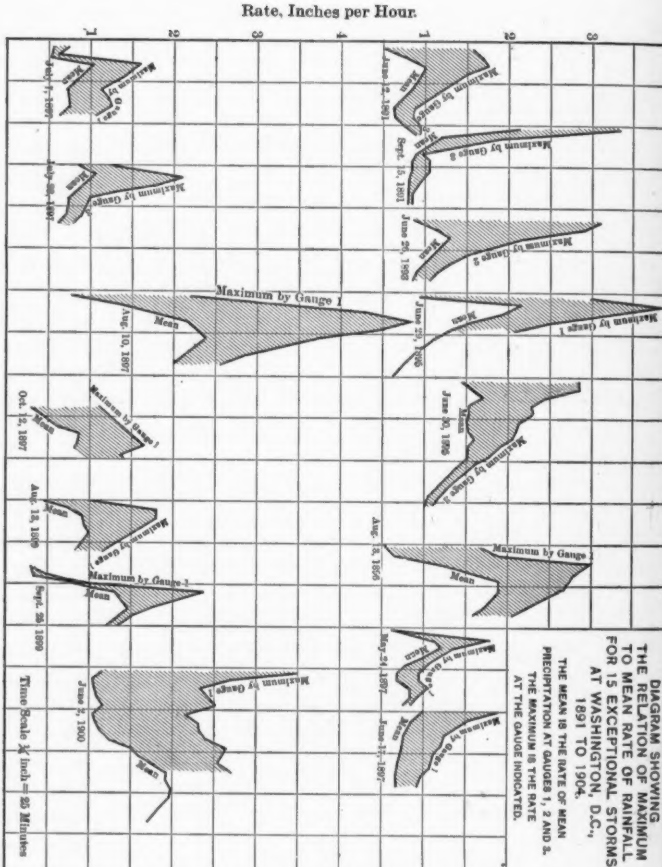
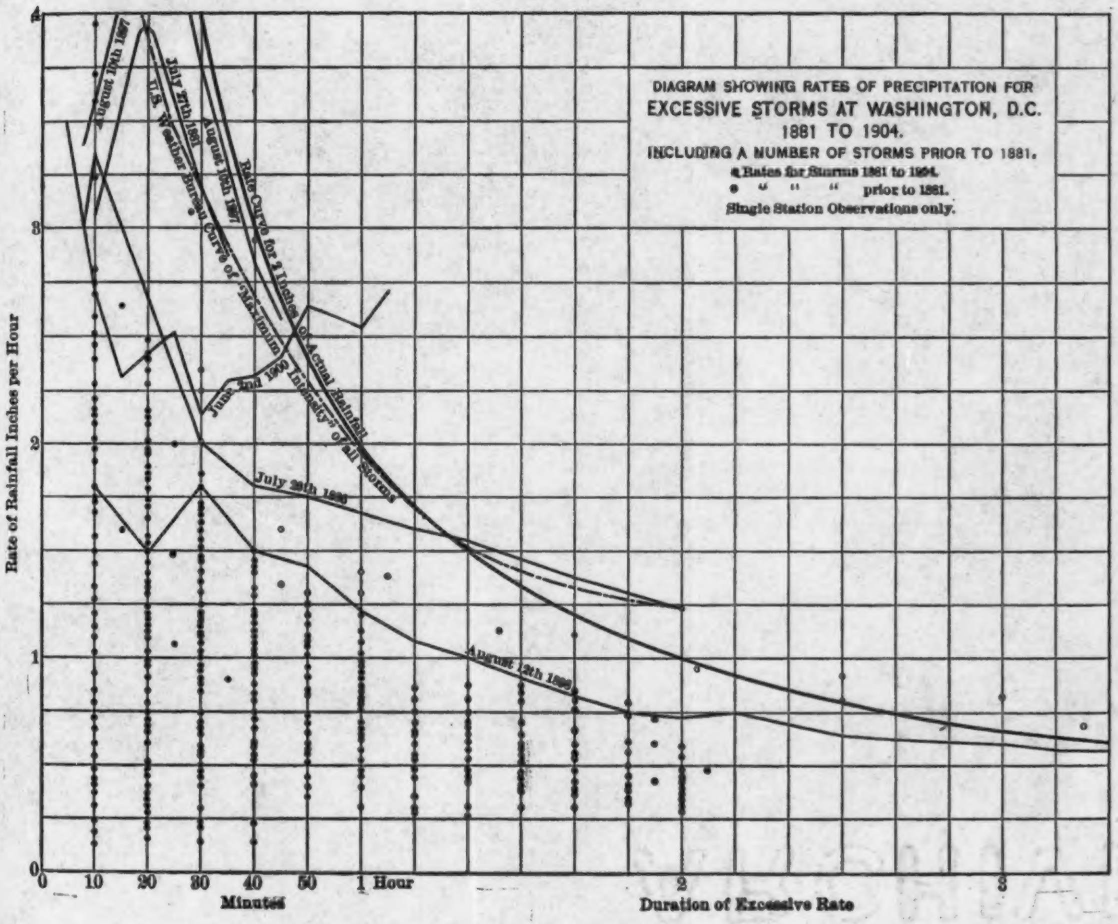


FIG. 7.

PLATE XVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 992.
PHILLIPS ON
MAXIMUM RATES OF RAINFALL.



1.—Comparative diagrams of certain exceptional storms, 1890-1904, as recorded at three stations in Washington, D. C. (Figs. 2, 3, 4, 5, 6 and 7); Mr. Phillips.

2.—Rate diagrams of several important storms (Plate XVII; also, rate points of local storms not otherwise shown; the 2-in. rate curve, and the Weather Bureau curve of maximum intensity, 1881-96 (Henry).

These records were obtained at gauges located as follows:

Gauge 1, at Twenty-fourth and M Streets;
Gauge 2, at Thirteenth and K Streets;
Gauge 3, at First and C Streets.

The distance between Gauges 1 and 2 is 2 100 yd., and between Gauges 2 and 3, 1 975 yd. The line of these gauges parallels the general river valley quite closely. The usual approach of intense storms is from the northwest, striking the gauges in their alignment and in their order as numbered. The gauges of the Engineer Department of the District of Columbia, designated as Gauges 2 and 3, were designed by Desmond Fitz Gerald, Past-President, Am. Soc. C. E., and are regularly inspected and very well maintained. The records plotted as of Gauge 1 refer to the gauge of the United States Weather Bureau, and are thus designated for convenience. They were obtained from the published records of the Bureau. Where single station records are plotted, the number of the gauge is indicated.

The data here presented are in part abstracted from a report (not heretofore published) by the writer, in 1900, on certain drainage works for the City of Washington.

This is not, perhaps, the first attempt to compare the records of a series of rain gauges covering a considerable area, and the results are altogether incomplete and tentative, yet this phase of the subject appears to merit discussion, and more extended observations may lead to a better understanding of several of the factors of run-off.

The feature of these comparative records, the writer would suggest, is not illustrated clearly by the curves of mean rate shown on the diagrams; for, in the latter, the variation in station rate is an important factor; but, rather, the very limited area of maximum precipitation, which is well indicated by the travel of the maximum across the field of observation.

Whether the great storms of 1898 at Philadelphia and Norfolk (Fig. 5), which were of extraordinary duration, were recorded correctly by the single station of observation, or whether the average rate for any period, over a considerable area, fell far below that record, it is not possible to determine. Local observations, however, seem to indicate that no storm has here been recorded having an average precipitation for the three stations exceeding 2 in. of

Mr. Phillips. actual rainfall in one hour, and no storm of great duration—and therefore considerable area, as city areas are measured—having an average which approached 2 in. in any one hour.

Mr. Kuichling. E. KUICHLING, M. AM. SOC. C. E.—This paper is a very valuable contribution to meteorology, and the author deserves great credit for his work. There is a point involved, however, which should be stated more clearly or definitely. This pertains to the meaning which is given to the term "intensity of downpour." It can be presumed, from the data submitted in Table 1, that the intensities recorded were uniform, or very nearly so, during the corresponding durations in minutes; but when the numerous long durations are observed, a doubt as to the correctness of such presumption arises in the minds of those who have studied closely the behavior of heavy precipitations, as it is rarely the case that the intensity is practically uniform for more than one hour.

To make the point under consideration somewhat clearer, take the case of the three heavy showers, each lasting from 20 to 30 min., and following each other in the course of 6 hours with a light rainfall or drizzle in the two intervals. The total precipitation may be 3.0 in., and the average 0.5 in. per hour, but this is very different from the uniform maximum, which may have been 2.0 in. per hour for three periods of 15 min. each. On one occasion, while in a mountainous district, the speaker observed five such alternating heavy showers and drizzles in a single afternoon. Rains of this description are often reported as being continuous, from which it may be inferred that they were also of uniform intensity throughout the stated period of time, thus leading to an erroneous conclusion.

A comparison of the three rainfall diagrams given in Fig. 1 with the corresponding numerical data given in Table 1, shows that on July 21st, 1894, the recorded intensity of 2.94 in. per hour during 22 min. was practically uniform; that on August 22d, 1899, the recorded intensity of 4.62 in. per hour during 22 min. was likewise nearly uniform, whereas the record of 1.65 in. per hour during 85 min. on the same day is not for uniform intensity, but is the average rate for two distinct, heavy showers with an interval of about ½ hour of drizzle; and lastly, that on September 20th, 1899, the recorded intensity of 1.07 in. per hour during 130 min. was nearly uniform, while the record of 0.44 in. per hour during 460 min. on the same day represents the average rate for the whole period, which includes about 6 hours of light rain and drizzle. As these three diagrams are given as samples, it may be inferred that the remaining records were treated in like manner by the author, and hence that all the data in Table 1 do not refer to uniform intensities.

Mr. Kuichling.

The conclusions reached from studies of the intensity of rainfall are usually applied in computing or estimating the maximum run-off from drainage basins of known area, and in many cases it becomes of great importance to determine such maximum without much room for error. This is particularly the case in the storm-water drainage of urban districts of moderate size, and as a large percentage of the precipitation usually finds its way quickly into the various tributary sewers or other drainage channels, it is necessary to know the probable maximum intensity of the rainfall for different periods of time. From the nature of the problem, however, a rational estimate of the proportion of the rainfall which flows off on the surface cannot be made unless the precipitation takes place at a uniform intensity on all parts of the water-shed; and hence, to be of good service, all compilations of maximum rainfall should be made with particular reference to uniform intensity during the period of observation.

The lack of such tabulations has led to many serious underestimates of the run-off in urban districts, and also to the establishment of many different formulas for computing the probable maximum flood discharge from river basins of greater or less magnitude. Usually, such formulas are made by platting as ordinates the greatest observed flood discharge, in cubic feet per second, from various catchment basins of similar topographical and geological character, but of different area, and the magnitudes of the basins in acres or square miles as abscissas; a curve is then drawn through or near the highest points in the diagram thus obtained, and, finally, the equation of such curve is sought in order to express the relation between maximum discharge and area. Further study of the data leads to the introduction of an assumed average rainfall intensity and the corresponding modification of the empirical constants of the original formulas, so as to make the expression applicable to some of the cases where the rainfall was known with more certainty.

The next step in the evolution of a flood discharge formula is to recognize in the expression the time needed for the surface drainage water to flow from the most distant parts of the catchment basin to the point of observation on the stream. This is often done by introducing the average slope or grade of the surface, as well as the average length and width of the territory. Here, again, trouble is encountered, owing to the perverseness of many of the data in refusing to fit the formula so laboriously devised; and in the hope of finding relief from the complication, the territory is then subdivided into two or three classes, such as mountainous, hilly and flat, the area of each class being introduced with a different coefficient or exponent, and also with corresponding different average grades. The results, hitherto, however, have not been found wholly satis-

Mr. Kuichling. factory, and when a fairly close agreement with a correct actual observation of flood discharge is encountered, such agreement may be attributed more to accident than to strict conformity with all the various factors involved. This is necessarily the case where a number of average values must be used.

In drainage work the principal elements to be considered are the intensity and duration of the precipitation, the extent and character of the surface upon which the water falls, with respect to absorption and evaporation, and the time which elapses before the run-off from the most distant points of the area reaches the point of observation. It is obvious that, when the rain continues sufficiently long at uniform intensity, the run-off from a given area will be the maximum corresponding to that particular intensity when all portions of the area are contributing to the flow at the same time. From every catchment basin, accordingly, there is a separate maximum run-off for every different uniform intensity of rainfall, and, in finding the absolute maximum, the only question is whether the higher intensities are of sufficient duration to allow water from all parts of the basin to reach the point of observation at the same time.

In the case of a large and uniformly constituted territory, it may happen that the absolute maximum run-off will occur at the end of a long rainfall of moderate uniform intensity. The reason for this fact is that in storms of short duration and high intensity, the flow from the lower portions of the area reaches the collecting stream and passes the point of observation before the flow from the upper portions reaches that point. It may also happen that, in a large but diversely constituted territory, the run-off will become greatest for a rainfall of high intensity and relatively short duration. This is due to the greater run-off from the less absorptive portion of the area than from the more absorptive remainder. On the other hand, the greatest run-off from a small area always occurs toward the end of a rainfall of high intensity. Drainage areas of the same general character in municipalities, therefore, may be classified according to the length of time taken for the surface water from the most distant parts of the district to reach the given point in the sewer or other watercourse. Thus there may be a 10-min. district, a 15-min. district, etc., from each of which the absolute maximum run-off will be governed mainly by its area and the corresponding uniform maximum intensity of the rainfall. Therefore, it is of the utmost importance to know the relation between this latter factor and its duration, and from the foregoing considerations it is plain that average intensities can be of no particular value in drainage computations.

From several recent compilations of heavy rainfalls of uniform

intensity, relating to localities in the vicinity of the city of New York, the speaker adopted the expression (using the author's notation): Mr. Kuichling.

$$i = \frac{120}{t + 20}$$

to represent the relation between the duration, t, in minutes, and the "ordinary" uniform maximum intensity, i, in inches per hour. If it were desired to construct a run-off formula with this expression as a factor, the process would be as follows: Let A = area, in acres; and m = run-off factor, or proportion of the rainfall which runs off from the surface at the time of maximum flow. This factor is generally the same as the proportion of impervious surface on the entire area, A. As the intensity, i, also represents very nearly the same number of cubic feet per acre per second, we would then have, if Q denotes the "ordinary" absolute maximum run-off, in cubic feet per second, from the area:

$$A : Q = m\ A\ i = m\ A\ \frac{120}{t + 20}.$$

As a matter of fact, however, the water which falls on a more or less irregular surface does not begin to flow off immediately, but accumulates thereon until the depressions are filled and the primary rapid absorption and evaporation have ceased. It then commences to run off slowly in a film or sheet of gradually increasing thickness until it reaches some kind of channel wherein it can flow more rapidly, and eventually it finds its way into some larger channel which conveys it quickly to the sewer or principal watercourse. Observation demonstrates that even in the heaviest showers several minutes elapse after the rain begins, before the water appears in appreciable quantity in the sewers or other collectors, and hence it is proper to increase t correspondingly in the foregoing expression. Usually, this is done by adding to t from 5 to 10 min., depending on the character or development of the given area, A. In general, therefore, the expression for Q can be put in the form:

$$Q = m\ A\ \frac{a}{t + b}.$$

If the area has the same character throughout, the factor, m, will be constant, but if the character changes in different portions, m will become variable as the area, A, increases. In municipal sewerage work, the factor, t, represents the number of minutes required for the passage of the water through the longest line of sewers in the district, when the same are full or nearly so; and as sizes, grades and velocities usually vary in considerable degree at different localities in the same area, the successive increments of t may correspond to widely different increments of A, so that, in

Mr. Kuichling. algebraic terms, A becomes a more or less complex function of t. For example, if two plane surfaces slope toward each other, so as to cause the velocity of the water which flows over the surface in a direction at right angles to the intersection of the planes to be equal to the velocity in the gutter formed by such intersection, the relation of A to t will be expressed by

$$A = n\,t\,(t + c).$$

Under other conditions we may have

$$A = n\,t^2,$$

$$\text{or } A = \sqrt{n\,t + p^2} - p,$$

$$\text{or } A = n\,t^2\,(1 - c\,t), \text{ etc.,}$$

depending on the peculiarities of the territory under consideration, and applicable only within certain limiting values of t.

Of these latter expressions, the first gives a curve which is convex to the axis of t; the second gives one which is concave to said axis; and the third gives one which is at first convex and afterward concave to said axis, but subsequently intersecting it at a distance $t = \frac{1}{c}$, so that A becomes a maximum for $t = \frac{2}{3c}$, whence the expression can obviously be valid only between the limits, $t = 0$ and $t = \frac{2}{3c}$. Furthermore, in both the first and the second expression, A never reaches a maximum for a finite value of t, and hence these forms can apply only to portions of basins of great magnitude. It is probable, therefore, that the third form is better adapted for use in a general formula relating to areas of moderate size, but the expression of t in terms of A then becomes very complicated, as it is the root of a cubic equation. For the sake of simplicity in further investigation, however, the value of Q in this case may be expressed in terms of t, thus obtaining:

$$Q = m\,a\,\frac{n\,t^2\,(1 - c\,t)}{t + b} = m\,n\,a\,\frac{t^2\,(1 - c\,t)}{t + b}.$$

In this general formula, Q becomes a maximum for

$$t = t_1 = \frac{1 - 3\,b\,c}{4\,c}\left[1 + \sqrt{1 + \frac{b}{c}\left(\frac{4\,c}{1 - 3\,b\,c}\right)^2}\,\right];$$

hence, if the area, A, has a practically uniform run-off factor, m, throughout, and is larger than A_1, corresponding to t_1, the maximum flood discharge will occur for a rainfall of maximum uniform intensity, i_1, corresponding to a duration of t_1 minutes, such duration being shorter than necessary to permit all portions of the entire area, A, to contribute at the same time to the discharge at the point

of observation at the foot of the territory. For example, assume that m is constant $= 0.5$, and that the relation between A and t can be expressed by $A = 0.5t^2(1-0.01t)$, giving **Mr. Kuichling.**

for....	$t = 10$	20	30	40	50	60	70 minutes,
the area	$A = 45$	160	315	480	625	720	735 acres;

also assume $i = \dfrac{120}{t + 20}$, and that the run-off is retarded 10 minutes, for the reasons mentioned above. We thus have $b = 20 + 10 = 30$, and $C = 0.01$, whence $t_1 = 57.3$ minutes, $A_1 = 701.2$ acres and $Q_1 = 482$ cu. ft. per sec., whereas if the entire area, $A = 735$ acres, and the entire time, $t = 70$ min., had been taken, the discharge would have been $Q = 441$ cu. ft. per sec. On the other hand, if the drainage area, A, were only 625 acres in extent, with its relation to t remaining the same as before, the maximum discharge would occur for a rainfall of maximum uniform intensity, i, corresponding to a duration of $t = 50$ min., or just sufficient to permit all portions of the territory to contribute at the same time to the run-off.

The foregoing example is similar to one which occurred in the speaker's practice, except that the difference between Q_1 and Q was considerably more than 10 per cent. It has been presented merely to illustrate a rational mode of procedure in the storm-water drainage of urban districts, and the necessity for maximum rainfall data based on uniform instead of average intensities. It may also be added that, while precipitation records of long duration and uniform intensity are rare, there are many such of short duration up to about 1 hour, and the tabulation of these for different localities will be of great service in municipal drainage operations, as it seldom happens that an urban district is so large and flat that the storm flow from its upper part to the final outfall will occupy more than 1 hour.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The intensity of rainfall for short periods of time is now being studied more carefully by weather observers, and, as a result, much valuable information is being obtained. **Mr. Le Conte.**

The older rainfall records at San Francisco are not by any means as complete as they might be, but, during later years, fortunately, the records of the United States Weather Bureau are available, and these are very much more in detail, and are quite reliable.

The older records were chiefly confined to San Francisco and Sacramento, and, while the results of intensity seem to be very meager when compared with those at Boston, nevertheless, they are equally interesting and important, from an engineering point of view.

Mr. Le Conte. The oldest intensity record, worthy of the name, is that of the noted rainstorm of December 19th and 20th, 1866. The intensity recorded was not extraordinary, *viz.*, 0.76 in. per hour, but the continuity, extending over 3 consecutive hours, is most remarkable for the Pacific Coast. Also, the total rainfall of 7.76 in. in 21 hours is phenomenal.

From this date until some time after the advent of the U. S. Weather Bureau, which began operations in March, 1871, no intensity records were taken; but intensity observations for shorter intervals of time, *i. e.*, for 1 hour and less, were not observed regularly previous to 1893. Since that date there is a tolerably fair record, which is full of interesting detail. It is thought best to put these results in tabular form (Table 3), as it is so much more convenient for reference.

TABLE 3.

Dates.	Rainfall, in inches.	Duration, in minutes.	Intensity, in inches per hour.
December 19, 1866	7.76	1 260	0.37
" " "	1.97	300	0.40
" " "	2.27	180	0.76
" " "	0.85	125	0.41
" " "	1.20	190	0.38
" 20, "	1.47	485	0.20
January 20, 1894	0.36	60	0.36
" " "	0.19	10	1.14
" " "	0.16	5	1.92
November 23, 1896	0.55	60	0.55
" " "	0.14	10	0.84
" " "	0.08	5	0.96
October 12, 1899	0.09	5	1.08
February 22, 1901	0.21	10	1.26
" " "	0.17	5	2.04
October 23, 1902	0.20	10	1.20
" " "	0.16	5	1.92
January 27, 1903	0.06	2	1.80
February 7, 1903	0.23	10	1.38
" " "	0.19	5	2.28

The data (in Table 3) are kindly furnished chiefly by Professor Alexander G. McAdie, of the U. S. Weather Bureau, San Francisco, Cal.

Perhaps it is a little too early to deduce a reliable intensity curve for San Francisco from the available data in Table 3, but, nevertheless, the writer found, on trial, that the curve follows very closely the simple expression, $i = \frac{7}{t^{0.5}}$, in which i = intensity of rainfall, in inches per hour, and t = duration, in minutes. For other cities along the Pacific Coast, it may be different, of course, and the writer would like to have the matter discussed.

WILLIAM MAYO VENABLE, ASSOC. M. AM. SOC. C. E.—In connection with this paper, reference might be made to a report presented to the Drainage Commission of New Orleans, La., some years ago by a special committee of engineers: Messrs. Rudolph Hering, B. M. Harrod, Henry B. Richardson and L. W. Brown, all Members of this Society. This was a report on the plan proposed for the drainage of New Orleans, which has since been partially carried into effect. It contains tables showing carefully collected data regarding the rainfall of New Orleans for a number of years. The report was printed, but the supply of copies for distribution has been exhausted. Nevertheless, the data are obtainable by those who are interested. Similar data have been obtained in that city since the publication of the report referred to and may be found in the records of the office of the Drainage Commission which was succeeded a year or more ago by the Sewerage and Water Board, of which George G. Earl, M. Am. Soc. C. E., is General Superintendent and Chief Engineer. Mr. Venable.

The speaker spent some years in New Orleans in the construction of drainage canals and pumping stations, representing the contractor who built the greater part of the work already finished on the drainage system, and thus had occasion to observe closely several very heavy rainfalls, and the rapidity with which the water found its way into the drainage system. In one storm, about two years ago, more than 8 in. of rain fell in 24 hours, and most of it fell in 3 or 4 hours. The city is almost flat, and especially in the rural parts, it takes the water considerable time to reach the canals, where it can be pumped.

The run-off is the controlling element in the design of drainage works where the surface is not paved, and especially where the canals or sewers are a considerable distance apart.

It would be very risky to base the design of a sewer for drainage purposes on general conditions of rainfall or of run-off, such as an average for the entire United States, because the factors to be taken into account are so various, and the damages to be sustained from the flooding caused by exceptional storms are so different in different places. For instance, the damage which might be caused by furnishing too small an outlet for an area where water would accumulate to any considerable depth is not comparable to that which might be caused in a comparatively flat territory, where the utmost injury that could occur would be the standing of a few inches on the surface for a few hours while the sewer was relieving the land of the excess.

During the summer of 1904, at Leavenworth, Kans., the speaker witnessed an exceptional rainfall. A culvert, serving to drain a considerable part of the city, proved to be too small, and, as a re-

Mr. Venable. sult, the water backed up and wrecked a number of dwelling-houses. The water came so suddenly, at night, that the people had no warning before the flood was upon them; and yet there had never been any flood there before, and the Missouri River was only a few hundred yards away, with a surface level of 15 or 20 ft. below.

Mr. Burns. C. S. BURNS, M. AM. SOC. C. E. (by letter).—The writer is pleased to note the painstaking manner in which the author has presented the data concerning the rainfall at Boston. It is to be hoped that additional data will follow in discussions concerning the converse problem, where data are available concerning the actual run-off over spillways passing the flood volume from known areas.

The writer has a record of an instance at Cherryvale, Kans., where the discharge over the spillway from a small storage reservoir would seem to indicate that the rate of rainfall must have been much in excess of that indicated by the theoretical curve, $i = \frac{25.12}{t^{0.687}}$ The records in the writer's possession are not from personal observation, but are measurements taken by the engineer in charge of the reservoir in question, and there is no reason to doubt their approximate accuracy. The reservoir was designed by the writer, therefore he is correct on the following points:

The area of the water-shed above the spillway is 1 283 acres. The spillway is designed so that the run-off can be computed with considerable accuracy for any known head over the weir. In the particular instance in question, the depth of water flowing over the weir, as indicated by the gauge, was reported by the engineer in charge as 3 ft. This, for the weir in question, gives a theoretical discharge of 1 860 cu. ft. per sec. The drainage area is slightly undulating, but comparatively level, and it is estimated from observation that the time of concentration of the run-off from the whole water-shed is approximately 2 hours. Taking, then, a storm of 2 hours' duration, the intensity of the rainfall, from the author's curve, would be approximately at the rate of 1 in. per hour. Water falling on the entire water-shed at the rate of 1 in. per hour would indicate a total rainfall of approximately 1 300 cu. ft. per sec., while the character of the water-shed in question would not lead one to anticipate greater than 50% run-off after making proper allowance for pondage and water otherwise held back or retarded. It seems, then, that the actual measured run-off in the instance cited was approximately three times the theoretical run-off computed from the author's formula, after making the usually accepted corrections for retardation of flow. In this particular instance, the excessive run-off cited was caused by the very first rain storm after the newly completed reservoir had been filled by several storms of minor intensity.

No harm was done, due to the fact that the main body of the dam was designed with a view to the ultimate raising of the spillway at least 4 ft. in case future requirements should make greater storage necessary. Mr. Burns.

S. WHINERY, M. AM. SOC. C. E. (by letter).—The importance of adopting and using appropriate and convenient units, in dealing with scientific and engineering data and computations, is universally admitted. As a measure of the intensity of rainfall, the unit which has been generally used by engineers is the rate of precipitation in inches per hour. This is, in some respects, an awkward and inconvenient unit, and the writer suggests that for it be substituted the actual depth, in inches, falling in the stated time. The measurement and designation of the quantity of rainfall, in inches of depth over the surface, is unobjectionable, and has become so well established that no change is desirable. But to designate the rate of precipitation, it would seem to be simpler and more satisfactory to refer it to the depth, in inches, falling in a given period of time. Thus, it is now the common way to say that for a period of 5 minutes rain fell at the rate of 3.6 in. per hour: It would be more direct and simple to say that for the period of 5 minutes the rainfall was 0.3 in. This is a direct expression, and the quantity is readily convertible into the common units of quantity of water, as, for instance, cubic feet per acre. This method of expressing rate of rainfall is the more desirable since the Government Weather Bureau reports of excessive precipitation are published in substantially this form, and we are largely dependent upon these reports for extended data on the subject, since the Bureau now maintains automatic recording rain gauges at a large number of stations. Mr. Whinery.

Diagrams of maximum rainfall constructed from records kept in this way show directly the quantity of water, in inches of depth, that may be expected to fall in any given time. Thus the author's curve, the equation for which is given as $i = \frac{38.64}{t^{0.687}}$ (Plate XV), transformed as here suggested, would appear as shown in Fig. 8, and the equation of the curve would become $d = \frac{38.64}{t^{0.687}} \times \frac{t}{60}$, where d = the total precipitation, in inches, falling in t minutes.

(This formula and the author's curve are erroneous for the shorter periods of time. There are no observations, so far as the writer knows, indicating a rainfall at Boston at the rate of 12.3 in. per hour for 5 min. The author's equation gives a rate of precipitation for 1 min. of 38.64 in. per hour, and as the time decreases the rate approaches infinity.)

Discussion of the possibility of constructing any formula or diagrammatic curve which will represent with approximate correct-

Mr. Whinery. ness the rate of rainfall at various places for various periods of time is probably more interesting in the abstract than useful practically, and it will become less useful as records of maximum rainfall become available for stations not very far apart, covering the whole country. But, in the absence of such full data, a formula which might be depended upon for reasonably correct results would undoubtedly be of value.

Perhaps, therefore, the writer may be excused for adding another attempt to devise such a formula. He has been under the impression heretofore that some rude relation probably existed between the rate of precipitation at any station and the normal annual rainfall at that station, but an examination of the records does not confirm that hypothesis. Thus, the curve for excessive precipitation, plotted from available records, for New Orleans, where the normal annual

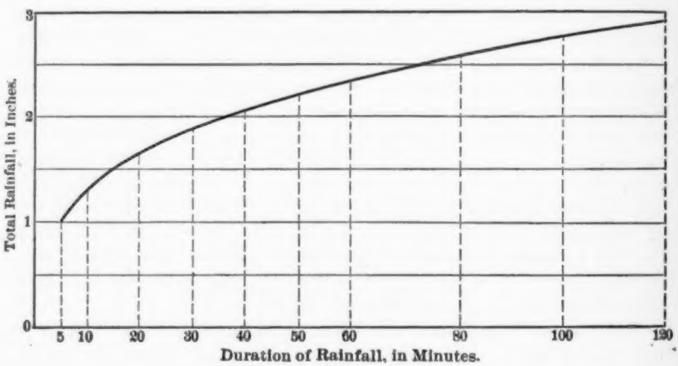


FIG. 8.

rainfall is 60.3 in., corresponds very closely with that for St. Paul, where the normal annual rainfall is 28.8 in.

It was for the purpose of testing this hypothesis that the records of excessive rainfall at nine cities widely distributed, geographically, over the eastern part of the United States, and having quite a wide range in annual rainfall, were collected and examined. These cities were:

	Normal Annual Rainfall.		Normal Annual Rainfall.		Normal Annual Rainfall.
Boston	45.4 in.	Atlanta	50.4 in.	Cincinnati	42.1 in.
Washington	42.9 "	New Orleans	60.3 "	Chicago	34.0 "
Savannah	50.4 "	St. Louis	40.8 "	St. Paul	28.8 "

The records of excessive precipitation at these stations, for various Mr. Whinery. periods of time up to 2 hours, were collected and tabulated (largely from the publications of the Government Weather Bureau, but supplemented by such other records as were available). A few (usually three or four) of the heaviest rainfalls at each station for the various periods of time were then diagrammed, and a curve constructed. These curves embraced all the recorded rainfalls for the various periods except a few very phenomenal downpours at some of the stations, not deemed likely to recur for long periods of time. In no case do the records of more than one rainstorm at any station lie without the curve, and then, usually, for but a few of the time-periods. The ordinates to the curve for each station were then tabulated, and the arithmetical mean for each period of time computed, and these means used as ordinates for plotting a mean curve. The original curves were found not to differ from each other very widely, when the apparently lawless character of rainstorms is considered. Table 4 shows, for each period of time, the maximum and minimum

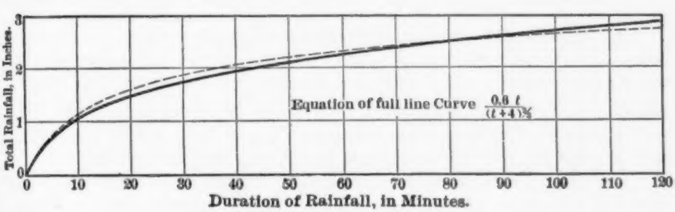


FIG. 9.

length of ordinate to any of the nine curves for the various periods of time, and their mean.

The curve shown by the broken line in Fig. 9 is platted with the mean ordinates from Table 4. This curve is somewhat irregular, and no equation exactly representing it was found. But a curve, the equation for which is

$$d = \frac{0.6\,t}{(t+4)^{\frac{2}{3}}};$$

in which d = the rainfall, in inches, for any number of minutes, t (up to 120 min.) coincides with it sufficiently close for practical purposes. This curve is shown by the full line in Fig. 9.

The equation is sufficiently simple to make its use reasonably convenient as a formula for excessive rainfall. It is recognized that neither the number of stations involved nor the time covered by the records warrants confident dependence upon its accuracy or reliability, but it is submitted as possibly more reliable than those heretofore proposed, and as giving results which may be safely used

Mr. Whirery. where satisfactory data based upon actual observations are not available.

TABLE 4.—MAXIMUM, MINIMUM AND MEAN OF ORDINATES TO EXCESSIVE RAINFALL CURVES IN NINE CITIES OF THE UNITED STATES.

Time, in minutes.	Maximum ordinate.	Minimum ordinate.	Mean of nine ordinates.
5	0.90	0.62	0.71
10	1.30	0.83	1.09
15	1.65	1.16	1.40
20	1.85	1.32	1.59
25	1.95	1.45	1.73
30	2.05	1.55	1.82
35	2.04	1.63	1.93
40	2.20	1.68	2.06
45	2.33	1.75	2.14
50	2.44	1.80	2.20
60	2.60	1.90	2.32
80	2.78	2.08	2.50
100	2.90	2.16	2.63
120	3.00	2.24	2.74

As intimated heretofore, results given by it would not include some of the phenomenal rainfalls in some of the cities, but if the coefficient of t in the numerator were increased from 0.6 to 0.7, nearly all these phenomenal storms would be covered, and the formula thus modified could be safely used, if it were thought wise by the engineer to provide against such phenomenal downpours.

The available records do not seem to be sufficient to warrant an attempt to apply the formula to periods of time beyond 2 hours. In fact, there seems to be good reason to believe that after that time the curve should deflect downward; in other words, anomalous as it may seem, the total precipitation in storms lasting, say, 4 hours, appears to be less than often occurs in storms of 2 hours' duration or less.

Mr. Webster. GEORGE S. WEBSTER, M. AM. SOC. C. E. (by letter).—In Mr. Sherman's paper on maximum rates of rainfall, comparison is made with several cities in the East, therefore it may be of some interest to add the information which has been collected by the Bureau of Surveys, Philadelphia.

In November, 1895, three self-registering pluviometers, of the Draper self-recording pattern, were placed in different parts of Philadelphia. In 1898 these were replaced by smaller, and more compact, machines, manufactured by Richards Brothers, Paris, to which others were added, making six in all.

Without attempting to show the variation of precipitation, during the same storm, in different parts of the city, the writer submits Table 5, which shows the rainfall for the 9 years during

which observations have been carried on, and embraces the principal storms. The curve of maximum rate of rainfall, determined from the plotted rates and durations for the same period, is shown on Plate XVIII. Mr. Webster.

TABLE 5.—PRINCIPAL RAINFALLS IN THE CITY OF PHILADELPHIA, FROM 1896 TO 1904, INCLUSIVE.

Date.	Total Rainfall.	Duration.		Maximum Rate per Hour.		
	Inches.	Hours.	Minutes.	Inches.	Hours.	Minutes.
1896—Jan. 24.....	1.12	11	00	0.18	..	..
Feb. 6.....	4.10	17	30	1.58	..	15
Mar. 6.....	1.18	13	00	0.27	1	00
May 3.....	1.09	1	25	0.92	..	10
June 14.....	2.35	11	30	0.43	1	00
July 6.....	0.52	2	00	3.40	..	05
" 29.....	0.34	1	00	3.00	..	05
1897—May 13.....	1.14	15	00	......	..	..
June 8- 9.....	2.25	21	00	1.25	..	15
" 19.....	0.99	3	00	1.80	..	05
July 21.....	1.33	13	00	1.80	..	10
" 22.....	1.56	8	00	1.80	..	10
" 23.....	0.99	4	00	1.98	..	10
" 27.....	1 5	24	00	0.48	1	..
Aug. 10.....	1.34	3	40	1.60	..	15
" 23.....	1.3	8	00	2.40	..	15
Oct. 12.....	0.44	2	00	2.76	..	05
Nov. 1.....	1.40	10	30	0.50	1	00
Dec. 14.....	2.20	19	00	0.26	1	00
1898—Jan. 19-20.....	2.45	29	00	......	..	..
May 7- 8.....	2.00	20	00	......	..	..
" 24.....	0.50	1	00	2.00	..	15
June 22.....	0.38	1	00	1.68	..	05
" 28.....	0.88	3	00	2.00	..	10
July 26.....	0.63	2	00	2.50	..	12
" 27.....	1.59	6	00	1.60	..	05
Aug. 3.....	5.48	7	00	{ 8.40 8.16	.. ..	04 05
" 3.....	5.48	7	00	3.56	1	00
Sept. 23.....	1.05	7	00	1.00	..	15
1899—Jan. 7.....	1.75	11	00	......	..	..
July 26.....	2.19	12	00	1.50	..	30
" 28.....	0.85	3	00	3.00	..	10
1900—Aug. 20.....	0.46	3	00	1.00	..	25
Sept. 16.....	4.50	12	00	1.80	1	00
Nov. 24.....	2.45	10	00	......	..	..
1901—April 3.....	1.31	17	00	......	..	..
" 24.....	1.18	23	00	......	..	..
Aug. 18.....	2.21	11	00	0.60	1	00
" 19.....	2.30	5	00	4.72	..	10
" 24.....	1.26	4	50	0.40	1	00
" 31.....	1.58	5	00	4.72	..	15
Sept. 11.....	1.70	2	50	4.68	..	05
Nov. 23.....	2.68	19	00	0.60	..	45
1901—Dec. 3.....	1.05	12	00	......	..	..
" 15.....	1.28	8	00	0.80	..	30
" 28-29.....	3.31	35	00	0.40	1	00
1902—Jan. 21.....	1.90	8	00	0.23	..	15
Feb. 21.....	1.60	8	00	0.40	1	00
" 28.....	1.80	5	00	2.72	..	15
Mar. 5.....	1.20	14	00	0.40	1	00
April 8.....	1.63	16	00	0.36	..	30
" 29.....	1.54	16	00	0.60	..	45
May 20.....	0.87	2	00	3.12	..	08
June 21.....	1.53	7	00	2.24	..	15
" 25.....	1.92	5	00	2.40	..	10
" 29.....	1.28	7	00	2.40	..	12
July 3.....	1.08	5	00	4.80	..	05
" 25.....	2.25	7	00	4.80	..	05
Aug. 10.....	2.05	5	00	2.40	..	15
Sept. 9.....	1.61	9	30	1.73	..	15
" 25-26.....	3.58	30	00	0.78	..	30
Oct. 1.....	2.51	12	00	1.20	..	20
" 5.....	1.90	20	00	1.12	..	15
" 28.....	1.57	9	00	0.90	..	12
Dec 21.....	2.19	15	00	1.18	..	20
1903—April 14.....	2.22	23	00	0.72	..	05
June 10.....	2.60	1	25	6.00	..	05
" 20.....	1.28	5	00	2.40	..	10
July 3.....	1.24	3	00	7.20	..	05
" 18.....	2.99	13	00	3.20	..	08
Aug. 4.....	1.30	7	45	3.10	..	03
" 7.....	0.50	2	00	4.20	..	05
" 28.....	2.35	17	00	1.00	..	10
Sept. 5.....	0.62	2	[illegible]	4.80	..	05
" 16.....	1.06	6	00	4.00	..	03
Oct. 8- 9.....	6.24	35	00	2.40	..	08
1904—Mar. 7.....	1.35	16	00	0.70	..	10
May 31.....	2.01	4	00	3.07	..	10
June 6.....	1.10	3	00	4.40	..	08
Aug. 10.....	2.53	7	00	4.00	..	05
" 20.....	3.56	9	00	3.34	..	10
Sept. 14.....	2.88	11	00	8.50	..	05
" 15.....	2.90	4	30	3.50	..	07
Oct. 12.....	1.54	11	00	0.70	..	10
" 21.....	3.06	7	30	3.14	..	03
Nov. 13.....	1.84	23	00	0.11	1	00

Mr. Webster. The heaviest rainfall recorded in Philadelphia occurred on August 3d, 1898, when in 7 hours there was a total fall of 5.41 in.; a fall of 1 in. in 12 min.; and a fall of 3.56 in. in one hour. This was local, and over the central portion of the city.

In the Frankford section, which is in the northeastern part of the city, and about 6 miles from the central portion, only 1.84 in. fell in 4 hours, with a maximum rate of 2.5 in. per hour, lasting 12 min.

In 1898 the total rainfall for the entire year was 49.40 in. or more than 9 in. greater than the normal, which, according to the United States Weather Bureau, is 40.02 in.

The Bureau of Water has obtained information as to the total rainfall for several years prior to the installation of their automatic gauges, and from data obtained from this source it is found that the average yearly rainfall between 1890 and 1904, inclusive, was 41.52 in.

From this and other sources it is found that, prior to 1896, the heavy rainfalls given in Table 6 have been recorded:

TABLE 6.

Aug. 12-13, 1872	5.21 in. in 24 hours.
Aug. 8-9, 1874	4.08 " " 24 "
Sept. 21-22-23, 1882	9.57 " " 48 "
	1.20 " " 25 minutes.
July 23, 1887	2.25 " " 1 hour.
July 30, 1889	3 " " 3 hours.
Aug. 14, 1889	1.54 " " 1 hour 20 minutes.
	1.15 " " 40 minutes.
Aug. 21, 1890	0.36 " for 5 minutes; rate, 4.32 in. per hour.
Aug. 21, 1890	0.50 " " 10 " " 3.00 " " "
May 28, 1894	0.48 " " 15 " " 1.92 " " "
May 28, 1894	0.61 " " 30 " " 1.22 " " "
Sept. 8, 1894	0.83 " " 30 " " 1 66 " " "

Table 6 shows the principal rainfalls in Philadelphia, in which the precipitation was greater than 1 in. per hour, or in which the maximum rate was high for short showers.

Figs. 10 to 16 are copies of precipitation records taken in different parts of Philadelphia.

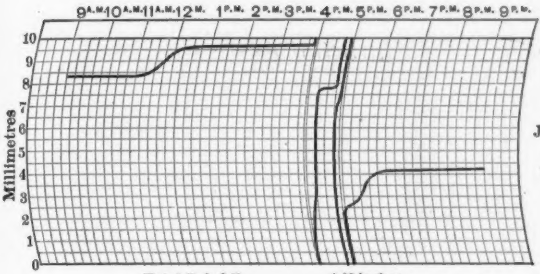
Plate XVIII shows the relation between the intensity of precipitation and the duration of rainfall for each of the storms included in Table 5.

In a number of these storms there were different rates of precipitation for different periods. Wherever the rate of precipitation is uniform for a certain period, the rate and duration are noted.

The upper curve, passing through the two points obtained from the records of the storm of August 3d, 1898, represents the maximum. The full line represents the curve of extraordinary rainfall.

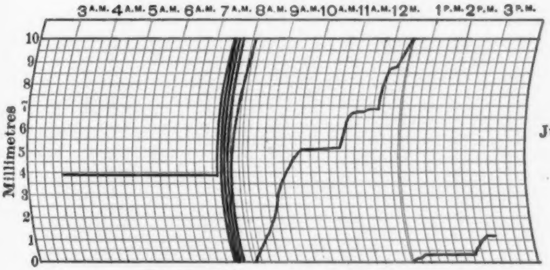
Mr. Webster.

PLUVIOMETER RECORDS OF STORMS IN SOUTH PHILADELPHIA.



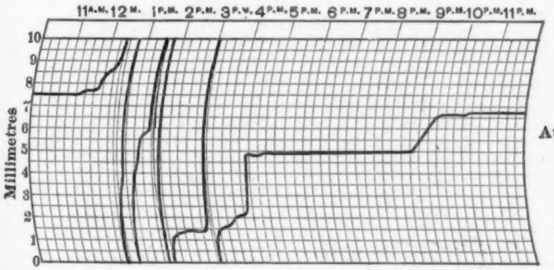
July 3rd, 1902.

Total Rainfall .... 1.08 inches.
Duration .... 5 hours.
Maximum rate per hour ... 4.8 in. for 5 min.



July 25th, 1902.

Total Rainfall .... 2.25 inches.
Duration .... 7 hours.
Maximum rate per hour ... 4.80 in. for 5 min.



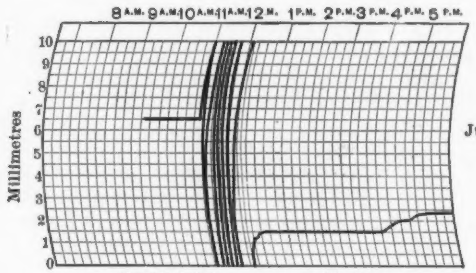
August 10th, 1902.

Total Rainfall .... 1.97 inches.
Duration .... 5 hours.
Maximum rate per hour ... 2.4 in. for 15 min.

FIGS. 10, 11 AND 12.

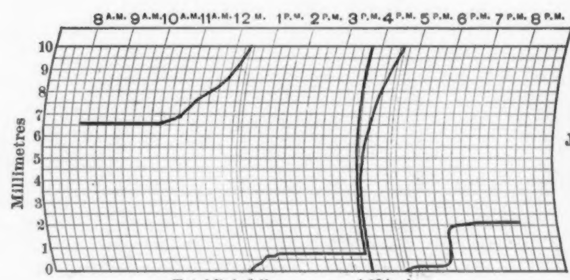
Mr. Webster.

PLUVIOMETER RECORDS OF STORMS IN SOUTH PHILADELPHIA.



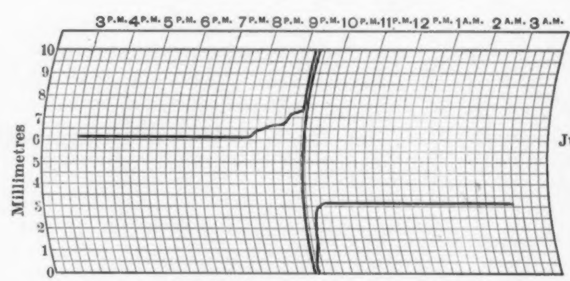
June 10th, 1903.

Total Rainfall.......2.60 inches.
Duration.........3 hours.
Maximum rate per hour...6.0 in. for 4 min.



June 14th, 1903.

Total Rainfall.......1.02 inches.
Duration.........7 hours.
Maximum rate per hour...4.5 in. for 5 min.

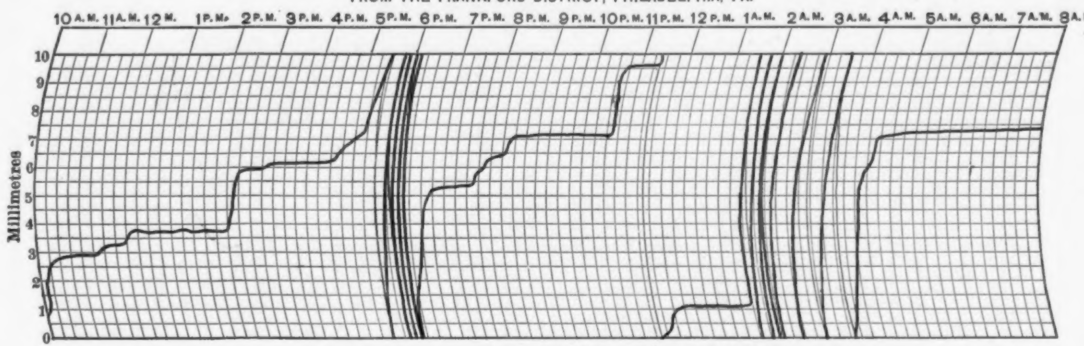


July 29th, 1904.

Total Rainfall.......0.67 inches.
Duration.........2 hours.
Maximum rate per hour...4.0 in. for 3 min.

FIGS. 13, 14 AND 15.

PLUVIOMETER RECORD
OF SEPTEMBER 14TH AND 15TH, 1904,
FROM THE FRANKFORD DISTRICT, PHILADELPHIA, PA.



	Sept. 14th	Sept. 15th
Total Rainfall	2.39 inches.	2.59 inches.
Duration	14 hours.	6 hours.
Maximum rate per hour	4.14 in. for 20 min.	5.5 in. for 5 min.
Rainfall in 1 hour	1.69 inches.	1.57 inches.

FIG. 16.

Mr. Webster. The broken line below this represents the curve of ordinary rainfall.

The question of run-off accompanying heavy precipitation is not touched upon in Mr. Sherman's paper. It may be noted, however, that the City of Philadelphia, for a number of years, has been gathering data by means of automatic stream gauges, and these data, taken in connection with the pluviometer records, have given valuable results in the preparation of designs for sewers.

As the importance of data of this kind necessarily depends upon the records of numerous storms which are scattered over a long period of years, no formula for run-off for general application has yet been determined.

Mr. Sherman. CHARLES W. SHERMAN, M. AM. SOC. C. E. (by letter).—It is very gratifying to note the interest in this subject, as shown by the number of discussions. The writer is under especial obligations to those who submitted additional data, as the subject is primarily a matter of records, while the conclusions to be drawn from them are simple.

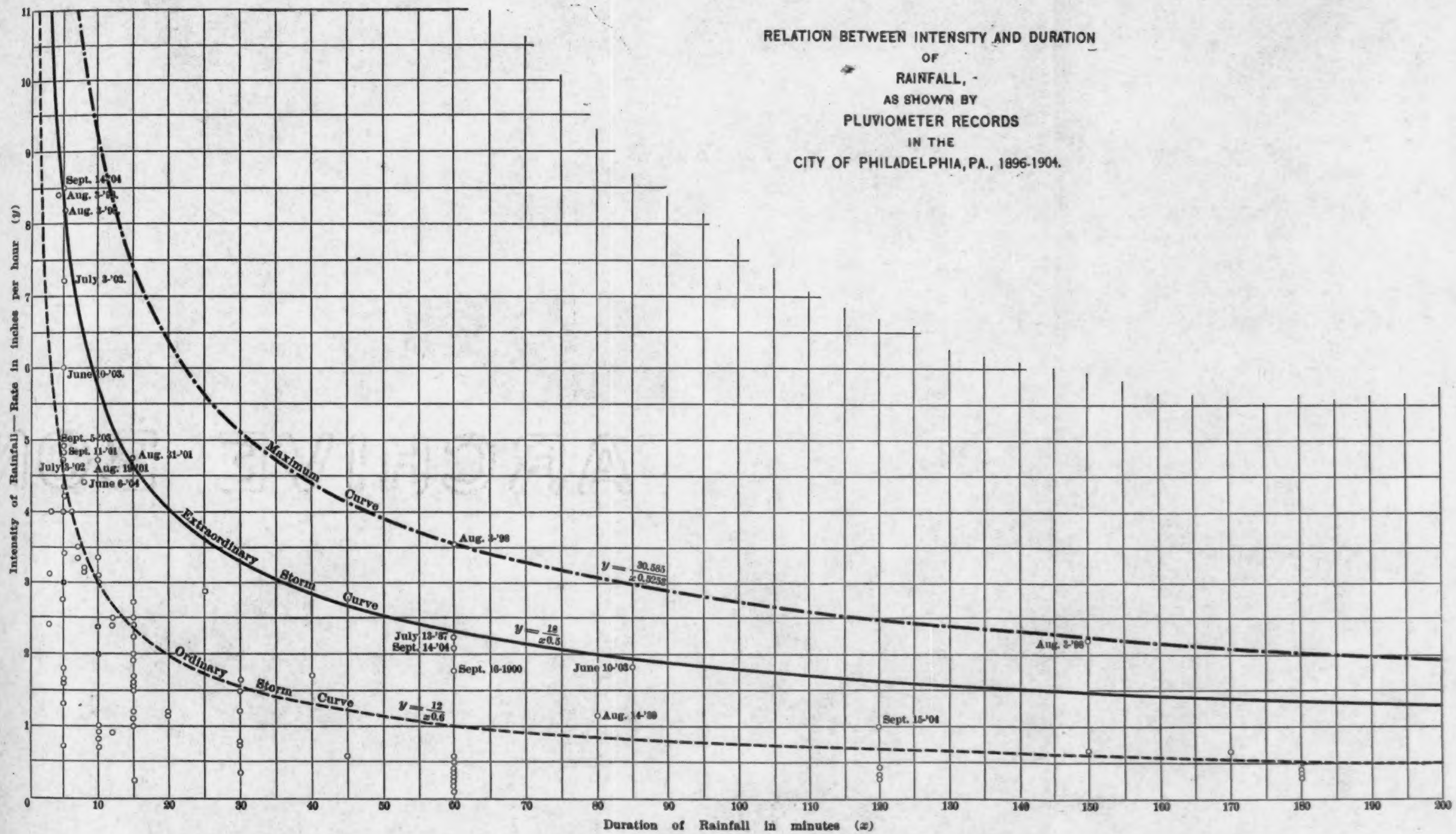
The paper did not discuss the subject of maximum run-off, as related to maximum rainfall, which is treated in some of the discussions. This is a different and far more complex subject, which the writer will not take up at this time.

Mr. Kuichling is right in his understanding of the term, "intensity of downpour," as used in the paper. For short periods, this is a correct term, the "downpour" being a portion of a storm when the intensity was large and substantially uniform. For longer periods, as a rule, the intensity was not altogether uniform, and the term should rather be "average intensity of rainfall" for the corresponding interval of time. It does not appear, however, that this should cause any difficulty in the use of the data, since in no case have been entered in this way storms which consisted of showers of heavy intensity and short duration connected with considerable intervals of light rain or "drizzle." The figures tabulated give the maximum average intensities of the storms for the periods of time stated; and where a shorter period is also tabulated for the same storm, as a rule, this is a part of the longer period during which the intensity was considerably greater and substantially uniform.

Mr. Whinery states that the form of equation given by the writer to represent the results obtained from the Chestnut Hill gauge is erroneous for the shorter period of time, in that the rate approaches infinity as the time decreases, and as no record has been obtained showing an intensity of 12.3 in. per hour for 5 min., which the "maximum" curve would indicate. It is well known, however, that for very short periods the rate of rainfall is often exceedingly high; and, since an intensity of 8.4 has been recorded for 5 min.,

PLATE XVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 992.
WEBSTER ON
MAXIMUM RATES OF RAINFALL.

RELATION BETWEEN INTENSITY AND DURATION
OF
RAINFALL,
AS SHOWN BY
PLUVIOMETER RECORDS
IN THE
CITY OF PHILADELPHIA, PA., 1896-1904.



and 16.4 for 1 min., it does not seem at all impossible that the rates Mr. Sherman. indicated by the "maximum" curve may not be attained for periods of 2 min. or more. Since, however, the absolute maximum rate of precipitation for intervals of time less than about 10 min. is of no practical importance, the exact form of the curve, as the duration becomes very small, is a matter of no moment.

Mr. Whinery refers to records of maximum rates of precipitation at several stations, mostly those of the United States Weather Bureau. A word of caution relative to the use of such records may not be out of place. Most of the Weather Bureau stations of importance, fitted with recording instruments, are located on the tops of Government buildings, usually from 50 to 150 ft., or even more, above the street level. The effect of such elevation of a rain gauge upon its collection has been investigated by Mr. FitzGerald and others. It is shown to some extent in Table 1, in which the ratio of the quantity collected by the recording gauge, elevated about 25 ft. above the ground, to that measured in a standard gauge close at hand, is seen to vary from 0.624 to 1.105 (for the entire storms), probably averaging about 0.88. The differences between the collections of elevated and surface gauges for the short periods of intense downpour are probably much more pronounced.

The data submitted by Mr. Webster are interesting and valuable. There appear to be some points plotted on his diagram, Plate XVIII, which are not included in Table 5. It would appear to the writer more logical to have the index of x the same in the three curve equations; this would result in curves differing somewhat from those shown, but very possibly representing the observations nearly or quite as well.

Mr. Webster's autographic records, Figs. 10 to 16, are on such a small scale that little can be made of them. The record in Fig. 16 appears to show a precipitation of nearly 9 mm. in no time or less.

Mr. Robert R. Evans, City Engineer of Haverhill, Mass., has recently reported* a storm of sufficient intensity to be worthy of record in this paper. It occurred on September 15th, 1904, and had the following intensities:

5	min.,	3.78	in.	per	hour.
10	"	3.65	"	"	"
15	"	3.40	"	"	"
30	"	2.88	"	"	"
60	"	2.10	"	"	"

An examination of the figures reported by Mr. Allen in Table 2 shows that some of them should be rejected unless supported by evidence not mentioned. The reports of the storm of August, 1843,

* City Engineer's Report, Haverhill, Mass., 1904.

Mr. Sherman. seem to be especially incredible, and unless reported by observers of known accuracy, with the method of measurement described, should certainly be thrown out.

Table 7 gives the maximum intensities authentically reported in the eastern part of the United States, as far as known to the writer.

TABLE 7.—MAXIMUM RATES OF RAINFALL AUTHENTICALLY REPORTED FOR THE EASTERN UNITED STATES.

Duration. Hours.	Duration. Minutes.	Rate. Inches per Hour.	Place.	Date.	Reported by:
0	1	16.40	Boston, Mass.	Aug. 1, 1885.	Sherman.
0	4	8.40	Philadelphia, Pa.	Aug. 3, 1898.	Webster.
0	5	8.16	" "	" 3, 1898.	"
0	5	8.50	" "	Sept. 14, 1904.	"
0	5	8.40	Boston, Mass.	July 18, 1884.	Sherman.
0	15	9.20	Embarras, Wis.	May, 1881.	Allen.
0	15	9.00	Sandusky, Ohio.	July, 1879.	"
0	20	6.78	Brattleboro, Vt.	July 7, 1897.	"
0	25	6.00	Kansas City, Mo.	May 12, 1886.	"
0	25	5.76	Indianapolis, Ind.	July, 1876.	"
0	30	5.60			Hoxie.
0	37	5.80			Talbot.
1	..	4.50			"
1	10	3.78			"
1	15	4.02			Nipher.
1	50	2.95	Philadelphia, Pa.	Aug. 3, 1898.	Henry.
2	00	3.00			Hoxie.
2	20	2.58	Newark, N. J.	Aug. 24, 1897.	Allen.
2	30	2.20	Philadelphia, Pa.	Aug. 3, 1898.	Webster.
3	00	2.15			Talbot.
3	20	2.05			"
5	..	1.80			Hoxie.
10	..	0.75	Wellsboro, N. Y.	May 31, June 1, 1889.	Allen.
12	..	0.77	Meridian, Miss.	Apr. 15-16, 1900.	"
18	..	0.82	Jewel, Md.	July 26-27, 1897.	"
24	..	0.57	Mayport, Fla.	Sept. 29, 1882.	"
36	..	0.34			Hoxie.
48	..	0.25			"
56	..	0.15			"

The writer had hoped that from Table 7 it would be possible to construct a simple curve which would represent at least fairly the heaviest intensities recorded in the eastern part of the country; but he has not been able to construct such a curve which would apply to both long and short periods of time. For periods of less than 3 hours, the expression, $i = \frac{420}{t + 30}$, expresses the maxima very satisfactorily, but, for longer periods, the intensities computed by this formula are considerably less than those given in Table 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 993.

ADDRESS AT THE ANNUAL CONVENTION AT CLEVELAND, OHIO, JUNE 20TH, 1905.

THE EVOLUTION OF THE PRACTICE OF AMERICAN BRIDGE BUILDING.

By CHARLES C. SCHNEIDER, PRESIDENT, AM. SOC. C. E.

In the time-honored custom of an address by the President at the Annual Convention of the American Society of Civil Engineers, many of my predecessors have selected topics with which they were most familiar. My life work naturally suggests to me the title of this address.

The magnificent stone bridges and aqueducts built by the Romans bear evidence that the art of bridge building had reached a high degree of perfection among the ancients. Some of the early Roman structures were of gigantic proportions, and have not been excelled by any of those built in modern times. There are no records to show that the ancients had any theoretical knowledge of bridge construction; they built their bridges in accordance with empirical rules developed by experience. These monuments of engineering skill, which have lasted for thousands of years, appear marvelous to the modern engineer, who has at his command, not only theoretical knowledge, but also mechanical appliances and

modern tools to assist him, which were not known in ancient times. The progress, therefore, made in the art of constructing bridges of stone in modern times appears insignificant, and is practically confined to theoretical knowledge and improvements in machinery and tools for handling material.

The development of the art of bridge construction is marked to a certain extent by periods coincident either with the introduction of new materials or with the necessities arising for means of transportation. The marvelous progress made in bridge building in the 19th century—the century of engineering, of the manufacture of power, of the railroad and the telegraph—really began with the production of wrought iron in large quantities, and has since kept pace with the development and progress made in the manufacture of those most useful and precious of all metals, wrought iron and steel.

The various types of bridges which have survived and become the standard types of the present day are the results of the evolution of a century, originating with the wooden bridge, the wooden beam being the prototype of the modern plate girder, and the framed wooden truss that of the steel truss. Timber bridges, consisting of hewed logs, supported by piers of piling or of stone, were erected in the earliest times in all countries where timber was abundant, Caesar's bridge across the Rhine was of this character. The wooden bridge consisting of framed trusses is the product of more modern times.

In Europe, wooden truss bridges of small span, patterned after existing roof trusses, have been built for several centuries. In America, however, it was not until the end of the 18th century that the movement began to which the present type of bridges can be traced.

There appear to be no records of any wooden bridges in America before 1785, when unusually gifted men like Palmer, Burr, Wernwag and others commenced to build some very remarkable wooden structures.

The progress made in bridge building in this country was very slow until railroad building commenced, which was in 1829, when the construction of the Baltimore and Ohio Railroad was begun. The first wooden railroad bridge was built on that road in 1830 by

Wernwag, at Monaguay. The development of railroads naturally created a demand for bridges.

The difficulties and obstacles encountered in crossing long and deep valleys were overcome by these pioneer railroad builders by erecting temporary timber trestles in place of expensive embankments or viaducts, to be filled in or rebuilt by permanent structures at a later period. The timber trestle, therefore, is a distinctly American type of construction, and is the prototype of the iron viaduct.

Some of the first high wooden trestles were built in 1840 on the Little Schuylkill and Susquehanna Railroad, now the Catawissa Branch of the Philadelphia and Reading Railroad. They were designed by James F. Smith, and their heights varied from 60 to 130 ft.

The forms of timber trusses of different kinds, arches and combinations of two or more systems, have been very numerous.

A marked step toward bridge designs of the modern truss form was the lattice bridge patented by Towne in 1820, which became the prototype of the early iron lattice bridge.

The next important step in the development of wooden bridges was made in 1840, when Howe patented his truss, which became very popular and the standard for wooden railroad bridges.

In 1844, the Pratt truss was patented, which afterward became the favored type for iron bridges. Many other types of trusses were invented, which have since been discarded.

The earliest wooden bridges were built by expert carpenters. The work was done by contract, very much the same as building work is done at the present day, except that the builder was also the designer. The builder would buy suitable timber or have it sawed to order at conveniently located saw-mills, and any ironwork needed in the construction of the bridge, such as rods, bolts or bars, he would obtain at a local blacksmith shop, and frame and erect the bridge in place, ready for traffic. The same methods were also used in building the early iron highway bridges. Each of these builders had his own type of bridge and his own special details. At that time there was generally but little competition, as very few had any knowledge of bridge building, and each one controlled a certain territory.

All the early railroad bridges were wooden structures built by practical carpenters, in most cases employees of the railroad company.

England is considered as the pioneer country of the iron bridge, the first one, consisting of a nearly semicircular cast-iron arch, having been built in 1776-79.

In 1786, Thomas Paine, the well-known author, designed and made a model of a segmental arch. This model was set up at Franklin's house in Philadelphia, whence it was taken to the State House, and, eventually, was sent to Paris and exhibited at the Academy of Sciences. Paine had an experimental cast-iron bridge built in England in 1790, and Rowland Burdon, in 1793 to 1796, built the bridge at Wearmouth, of 240 ft. clear span, after this model, which formed the basis of many cast-iron bridges built thereafter, and became the prototype of the modern steel arch. Paine's device was also the basis of the design of the Market Street Bridge and the first Fairmount Bridge, in Philadelphia, both being wooden arches. The former was completed in 1800, and the latter in 1812.

Up to 1840, there were no iron bridges in this country, except suspension bridges in which iron links were used in the cables and suspenders, the floor-system being of wood.

The first bridge in America consisting of iron throughout was built in 1840 by Earl Trumbull over the Erie Canal, in the Village of Frankfort, N. Y. In the same year Squire Whipple, Hon. M. Am. Soc. C. E., also built his first iron truss bridge.

Probably the first iron railroad bridge was built on the Philadelphia and Reading Railroad at Manayunk by Richard B. Osborne, Chief Engineer, in 1845. It was a double-track through bridge, of 34 ft. clear span, of the Howe truss type, with cast-iron top chord and web braces, the bottom chord and vertical web members being of wrought iron. This bridge was followed by several others of the same type.

The earlier wooden and iron bridges were built very much in the same manner as the ancient Roman bridges, in accordance with empirical rules, by practical men who had no accurate knowledge of the strains produced on the various members of a structure by the exterior forces, but who were men of unusual constructive ability

and sound judgment, who had to depend upon their own resources and natural instinct, experimenting with models and profiting by previous failures. Practice always preceded the science, thus the structural systems were invented before their theory was developed.

Until 1847, when Squire Whipple, the modest mathematical instrument maker, who, without precedent or example, evolved the scientific basis of bridge building in America, correct methods of computing the strains in framed structures were not known. A few years later, in 1851, Herman Haupt published a book on the theory of bridge construction.

About 1850, after the building of railroads had advanced, the educated engineer commenced to exert his influence in the art of bridge building, and, from that time forward, steady progress was made.

The period from 1850 to 1860, therefore, may be regarded as an epoch in the history of American bridge building; the time when the bridges designed by Fink and Bollman first came into use, and the earliest iron Whipple and Pratt trusses were built.

When American engineers commenced to build iron bridges, they paid little attention to the then existing European models, but preferred to develop their own systems independently, as they had done previously with wooden bridges, the first iron bridges being imitations of the Towne lattice, and the Howe and Pratt trusses.

All the earlier bridges were built principally of cast iron, wrought iron being used in tension members only. In the first iron viaduct built by the Baltimore and Ohio Railroad, in 1852, all parts were of cast iron, except the tie-rods. The wrought-iron tension members at that time usually consisted of round bars with screw ends, or elongated links made of square bars. Later, these links developed into forged eye-bars, introduced by J. H. Linville, M. Am. Soc. C. E., in 1861. These eye-bars have since become one of the distinctive features in American bridge construction. Although flat eye-bars were used in Europe at an earlier period, in chains of suspension bridges and in some types of trusses, they did not find favor there, and were soon discarded for structures with riveted connections.

The first bridges made entirely of wrought iron were those of

the riveted lattice type which Howard Carroll, then Assistant to George E. Gray, Hon. M. Am. Soc. C. E., Chief Engineer of the New York Central Railroad, commenced to build in 1859; next came the plate-girder type, the first of which was built by E. S. Philbrick, M. Am. Soc. C. E., for the Boston and Albany Railroad in 1860.

The bridge built by J. W. Murphy in 1863, over the Lehigh River at Mauch Chunk, for the Lehigh Valley Railroad, was the first pin-connected bridge constructed entirely of wrought iron in its main members; cast iron being used only for joint boxes connecting the compression members.

Many bridges of similar construction were built after this, but it was not until after the failure of the Ashtabula Bridge, in 1876, that cast iron was entirely discarded as too unreliable a material to be used in any parts of a railroad bridge.

Prior to 1860, railroad bridges were generally designed by the railroad companies' engineers, the ironwork being manufactured at the companies' shops, and erected by their own forces.

Thus, men like Wendell Bollman, Albert Fink, Past-President, Am. Soc. C. E.; C. Shaler Smith, M. Am. Soc. C. E., and C. H. Latrobe, M. Am. Soc. C. E., on the Baltimore and Ohio Railroad; Richard B. Osborn and Charles Macdonald, M. Am. Soc. C. E., on the Philadelphia and Reading Railroad; J. H. Linville, on the Pennsylvania Railroad; E. S. Philbrick, on the Boston and Albany Railroad; George E. Gray, Howard Carroll and Charles Hilton, on the New York Central Railroad; Williard S. Pope, M. Am. Soc. C. E., on the Chicago and Northwestern Railroad; Thomas C. Clarke, Past-President, Am. Soc. C. E., on the Chicago, Burlington and Quincy Railroad; S. S. Post, M. Am. Soc. C. E., on the Erie Railroad, were prominent railroad engineers who took a leading part in early bridge building.

Later, some of the men who had gained experience in framing and erecting bridges, or in the construction of the work at the shops, started in business for themselves, and took contracts to build and erect bridges on designs furnished by the railroad companies' engineers. Most of those early firms were contractors for building Howe truss bridges, only a small shop being required to manufacture the ironwork needed for structures of that class.

Some of the bridge engineers employed on railroads, seeing that

they could use their knowledge to better advantage in the more profitable business of contracting, associated themselves with the then existing bridge building firms, or organized new companies. These new companies often made a specialty of manufacturing constructions of a certain type, expressing the individuality of the engineer at their head, and which were his own inventions, in many cases controlled by patents. They were able to furnish designs for bridges, as well as construct and erect them.

Most of those companies were organized between 1860 and 1870, which period, therefore, forms another epoch in the history of American bridge building.

Near the end of the Sixties, when most of the early bridge companies had been formed, there were, besides the engineers interested in bridge building firms, only a few experienced bridge engineers in this country. The engineers who were at that time connected with bridge companies were mostly men who had gained their experience in the employ of some railroad company, had worked out their own type of construction, and had experience, not only in designing, but also in superintending the construction and erection of bridgework. Their theoretical knowledge, measured with the present standard, was limited to elementary methods, but their thorough practical training enabled them to combine theory and practice to the best advantage. They understood how to make their designs conform to the methods of the workshop, as well as to facilitate erection.

This was really the beginning of the development of American bridge building and of the distinctly American types of construction which at that time differed so materially from those of other countries.

The most distinguishing feature of the methods then prevailing in this country, as compared with those of other countries, the influence of which is felt to the present day, is that at that time in America the bridges were designed by experienced specialists, and the work was constructed in shops built and equipped for that special purpose by experienced mechanics trained in that class of work.

At first these companies controlled the work in certain territories, or the contracts were awarded to them on account of the reputation of their engineer.

However, as competition became keener, railroads desired to purchase their bridges for the lowest price, and invited several firms or companies to submit tenders on the bidders' own designs, which started the competitive system of designing and bidding on bridgework.

Up to about 1872, specifications, as we now understand the term, were not in general use. An invitation to bid on bridgework would be accompanied by a survey plan, giving the length of the spans, skews, etc., and a statement of the live load the bridge was to carry, generally a uniform load of 1 ton per linear foot, with a factor of safety of 5.

The design of the structure and the proportioning of its members and their details and connections were left entirely to the judgment of the builder, and accepted without question by the purchaser. These builders, who at that time were about the only bridge experts in America, were considered authorities, and would assume the responsibility for the design and the strength of the bridge for the specified loading. In other words, the bridges were accepted on faith and on the strength of the reputation of the builder.

From 1864 to about 1874, the designing of bridges was almost entirely in the hands of the bridge-building firms, only a few railroads, such as the Pennsylvania, and Boston and Albany, employing their own bridge engineers to prepare the designs and to supervise the construction of bridges and other structural work on their respective roads. The contracts for bridges were then let on a lump-sum price for the work erected in place. It was therefore to the interest of the bridge builder to make designs which would reduce the cost of shop work, as well as that of erection, and, as the price of wrought iron was high, as compared with the price of steel at the present time, the saving of material was one of the most important considerations in the designing of bridges.

At that time railroad construction had commenced to develop rapidly in all parts of the country, and many wide and treacherous rivers had to be spanned with bridges. The material for these bridges had to be transported to distant places, erected in unsettled locations, and the ironwork manufactured and erected rapidly in order to keep pace with the swift progress made in the building of railroads.

As the pin-connected type fulfilled, more than any other, these

requirements, *viz.*, economy in weight and facility in manufacture and erection, thereby not only reducing the cost, but also the risks and dangers of erection, it became the favorite type of bridge, and has remained so for long spans to the present day.

The first more comprehensive specifications were those published by Clarke, Reeves and Company, in 1871. In 1873, George S. Morison, Past-President, Am. Soc. C. E., prepared specifications for the Erie Railroad, which were probably the first printed specifications for iron bridges adopted by any American railroad. In 1875, L. F. G. Bouscaren, M. Am. Soc. C. E., wrote specifications for the Cincinnati Southern Railway, the first in which concentrated wheel loads were specified for the live load. Mr. Morison established the practise, on the Erie Railroad, of requiring the successful bidder to submit strain sheets and plans for approval before ordering material or commencing work. These plans, before being approved by the chief engineer, were examined by one of the assistant engineers, and, later, the material and workmanship were inspected by men in the employ of the railroad. The same practice was also adopted by Mr. Bouscaren on the Cincinnati Southern Railway.

The year 1873, therefore, may be considered as marking the beginning of another epoch, *viz.*, that of the bridge engineer as again acting in the interests of the railroad company, and also the beginning of the inspection of structural ironwork. Before that date the St. Louis Bridge was probably the only case on the construction of which inspectors of material and workmanship were employed.

Many new bridge companies were established after 1874, some of which had a marked influence in the development of American bridge construction, as well as in the improvement of tools and machinery, resulting in a higher grade of work.

Some of the engineers who had gained experience in the employ of bridge companies obtained positions on railroads as bridge engineers; others, particularly the most competent, left the employ of bridge companies for private practice, and devoted themselves to the specialty of structural engineering, acting in the interests of corporations and municipalities or other purchasers of iron structures.

These corporations commenced to realize that it would be to their advantage to have the services and advice of competent specialists. Thus, in 1876, the existence of the bridge engineer and

structural expert, independent of a bridge company, had become an established fact.

After iron railroad bridges had been in service for about twenty years, engineers who had charge of their maintenance noticed that weak points developed under traffic, particularly in the details and connections. It also became apparent that the bridges built, up to about 1875, were deficient in rigidity and lateral stability, and improvements were gradually made to remedy these defects, producing more massive construction, fewer and heavier parts, and a more extensive use of riveted connections.

The pin-connected type of truss for short spans was gradually discarded, the plate girder and riveted truss taking its place, and the limiting length of spans for these types was gradually increased. Specifications for iron bridges were also revised and improved; those prepared in 1877 by Charles Hilton for the Lake Shore and Michigan Southern, and by C. Shaler Smith for the Chicago, Milwaukee and St. Paul Railroad, and in 1879 by Theodore Cooper for the Erie Railroad, being steps in that direction.

Steel, as a structural material, was first used in a portion of the St. Louis Bridge, completed in 1874, but the first bridge built entirely of steel was the Glasgow Bridge, over the Missouri River, completed in 1879.

The extensive use of steel, however, did not commence until 1890. Before that time steel was used only in isolated cases, or for heavy work, such as chords and eye-bars for large spans.

About 1890, some railroads commenced to build also smaller spans and plate girders of steel, and, for eye-bars, steel was almost exclusively used. At that time most of the rolling mills, which had formerly manufactured wrought iron, were equipped with steel furnaces, but continued for some time to make both kinds of material, until they found it more profitable to confine themselves to the manufacture of structural steel only, and discontinued the manufacture of wrought iron. In 1894, it was practically impossible to obtain wrought iron shapes, and from that time forward steel entirely superseded wrought iron as the modern structural material. The year 1894, therefore, may be considered as the commencement of the present epoch—the steel age.

The use of steel in the construction of modern bridges, and the

improvements made in its manufacture, thereby reducing its cost and increasing its reliability, have made it practicable to build structures of a magnitude never attempted before.

Its introduction as a structural material had a marked influence on the progress and development of bridge building. Certain types of construction and details which have proved unsatisfactory have been discarded, and others which have undergone a process of purification and improvement have survived. Rational types of construction and details are now established, and have become the recognized standards for ordinary bridges of moderate spans. The present tendency is toward uniformity, and to-day there is but little difference between the designs made by competent engineers.

This tendency toward uniformity has also been extended to specifications for bridges and other steel structures, relating to quality of material, workmanship and unit strains.

As late as five years ago, the requirements specified for the quality of steel were numerous, almost every railroad or bridge engineer requiring some different grade, and sometimes several different kinds in different parts of the same structure.

Erratic specifications are now gradually disappearing, and engineers at the present time are nearly all agreed on the grade of steel best suited for structural work.

There is also at present more uniformity between the designs made by American engineers and those by European engineers. In the early days of iron bridge building, in this country, there was little resemblance between American and European structures. Each country gradually adopted the good points of the other's practice; we adopted their practice in the use of riveted trusses for longer spans and a more extensive use of riveted connections; while the European engineers are adopting the more rational designs and details of plate-girder and riveted-truss construction now used in America.

At the present time, the designs of plate girders and ordinary riveted-truss bridges, made in this country, are almost identical with the designs made by the best bridge engineers in Europe, so that no vital difference now exists between American and European bridges of moderate spans.

The steady increase in the weights of locomotives and rolling stock has been the cause of constant replacements of iron and steel railroad bridges by heavier structures. As the extreme limit of loads may not yet have been reached, the probable future increase should be anticipated in designing new bridges which have to carry any kind of railroad traffic. While it is impracticable to provide for all possible emergencies, railroad bridges should be designed to withstand the ordinary contingencies of traffic, such as derailment, a broken axle or a collision on the bridge. Structures designed in accordance with good practice may be damaged by such accidents, but should be able to stand up without collapsing.

As steel is practically an indestructible material, if kept from corrosion, there is no good reason why properly designed steel bridges, properly protected, should not last at least as long as stone bridges in this climate.

About 1886, a new type of iron structure came into existence, *viz.*, the iron skeleton construction for buildings, which has opened a new field for the structural engineer. The designing and construction of the structural part of these buildings has now become an important branch of engineering.

Any engineer who has followed the progress of American bridge building for the last 35 years must have observed that, not only the designs, but also the methods adopted for accomplishing results, have undergone a vast transformation; while, abroad, the designs have been improved, but the methods have changed very little, if at all.

The practice of having the designs of bridges and other structures made by engineers employed by the purchaser, and letting the contract to a manufacturer on a pound-price basis is now becoming the standard practice of the country. Most of the large railroads have their own bridge and structural departments, or, for work of unusual magnitude, employ outside experts to design and supervise construction. Only a few of the smallest railroads adhere to the ancient practice of inviting manufacturers to submit competitive designs accompanied by a lump-sum bid.

The competitive system has had its day and has served a good purpose. It has been an important factor in developing the art of bridge building in America. It has been productive of establishing rational types, practical details and scientific proportions; it

has united theory and practice. However, at present, it is fast becoming a thing of the past.

What is left of this practice is mostly confined to bridges for electric railways and light structural work. Many purchasers of structural work, who have had no experience themselves, do not seek professional advice, as they would in other cases where large expenditures of money are involved, believing that they can save the money paid for professional services by inviting manufacturers to make competitive designs accompanied by a lump-sum bid.

The fact is, however, that the manufacturer has to pay for making the designs not only once, but many times over, as only once in a number of cases he is the successful bidder. The manufacturer will naturally add this extra expense to the cost of the structure, yet the designs are not made in the interest of the purchaser, but in that of the manufacturer. This practice has a demoralizing influence, as it puts a premium on the poorest design and tends to decrease the professional standard of an important branch of engineering.

The standard practice to be recommended, as the only fair and business-like method, is to let contracts for structural steelwork on a pound-price basis, on designs and specifications furnished by an experienced engineer employed by the purchaser. This method is fair to the honest manufacturer, as all competitors bid on the same basis; it is an advantage to the purchaser, as he employs the engineer who will protect his interests, study the conditions and requirements, and design a structure to suit the needs of his client. It will benefit a number of engineers, who are now compelled to waste their time and energies in making speculative designs to suit the commercial interests of a manufacturer, regardless of good practice, by elevating them to more independent positions, thus enabling them to raise their professional standards to the highest ideals of good practice.

Plans for bridges and other structures, on the safety of which the lives of human beings depend, should be designed and not manufactured; their design and the supervision of their construction should be entrusted only to competent engineers, who, besides the requisite theoretical and practical knowledge, should, above all, be endowed with common sense and good practical judgment.

Most of the largest bridges and other steel structures which have been built in later years have been designed by engineers not connected with manufacturing establishments.

The manufacturer should confine himself to his legitimate field of manufacturing structural steelwork at so much a pound.

The line between engineers and manufacturers will be even more marked in the future, when the same distinction will prevail as now exists between the architect and the contractor. The manufacturers of structural work, in the future, will devote their energies to improvements in their tools and machinery and methods for handling material.

Their engineering force will consist of mechanical experts, shop draftsmen and engineers, who, with a thorough knowledge of shop-practice, are skilled in putting the engineers' designs into convenient shape for the workshop.

Patents on structural designs and details, as well as on special shapes, have become unpopular. Designs of important structures, or those with new features, are now generally published for the benefit of the profession, and each engineer endeavors to improve upon the design of the other.

Beneficial results are derived from papers, submitted to this Society, illustrating and describing new structures, as they bring out valuable discussions and thereby advance engineering knowledge.

Highway Bridges.

Highway bridges in large cities are, at the present time, generally designed by experienced engineers, and the contracts are let in accordance with legitimate practice, the material and workmanship receiving the same careful inspection and supervision as railroad bridges.

These structures, however, do not represent the general run of highway bridges throughout the country. In 1852, Squire Whipple stated, in a pamphlet published by him, entitled, "The Canal Bridges, a Specimen of the Manner of Awarding Contracts by the late Canal Board," that the highway bridges over the New York State canals were let to the highest bidders and on the poorest designs submitted. The same conditions exist to some extent at the present day.

These bridges are frequently designed by incompetent or unscrupulous men, and the contracts are awarded by ignorant county officials, without the advice of a competent engineer. The merit of the design receives generally no consideration, and the contract is awarded in many cases to the one offering the poorest design and making a bid which is satisfactory to the officials, if not to the taxpayers.

This condition will probably continue until, after repeated disasters, the public demands that competent engineers design and supervise the construction of county highway bridges, and that the contracts be let in accordance with legitimate practice.

Long-Span Bridges.

Long-span bridges have, of late, not only become more numerous, but their length has been gradually increased, until the long-span bridges of former years are now considered spans of moderate size only. The necessity for the great number of long-span bridges in the United States arose from the fact that many wide navigable rivers had to be bridged where the interests of navigation demanded long spans, or where they were required on account of deep and expensive foundations, or on account of other conditions determining the length of spans. There are now in existence in America about fifty railroad bridges containing simple spans of 400 ft. or more, eighteen of which exceed 500 ft., the longest one being the Ohio River Bridge at Louisville, with a span of 546 ft., completed in 1894; there are also a number of highway bridges exceeding 400 ft. span. The length of spans practicable for simple trusses has not yet been reached.

Suspension Bridges.

Suspension bridges were in use in America before any other type of iron bridges. In the earliest suspension bridges, wrought-iron links were used for the cables, and a wooden floor system was suspended from them by iron rods. The first of this kind was built by Finley, in 1796, over Jacob's Creek, on the turnpike between Uniontown and Greensburg, Fayette County, Pa. Many other bridges of the same type were built by Finley after this, the largest one being that over the Schuylkill River, at Philadelphia, Pa., of

306 ft. span, built in 1809. The first wire-cable suspension bridge, was also built over the Schuylkill River, at the Falls in Philadelphia, Pa., in 1816; the span was 408 ft. This bridge had a wooden floor system and no stiffening trusses. After this, the wire-cable suspension bridge with auxiliary stiffening trusses became the favorite type for long-span highway bridges, owing to the facility of its erection. Many famous long-span bridges have been constructed of this type, such as:

The bridge over the Ohio River, at Wheeling, 1 010 ft. span, built in 1855.

The bridge over the Niagara River, at Lewiston, 1 040 ft. span, completed in 1850.

The bridge over the Niagara River, carrying the Grand Trunk Railway and the highway, 821 ft. span, completed in 1855. This bridge has become famous as being the only wire-cable suspension bridge carrying highway as well as railroad traffic.

The bridge over the Ohio River, at Cincinnati, 1 000 ft. span, completed in 1867.

The bridge across the Niagara River, below the Falls, at Clifton, 1 264 ft. span, finished in 1867.

The most notable suspension bridges, however, are the Brooklyn and Williamsburg Bridges, across the East River, New York City.

The Brooklyn Bridge, 1 595 ft. span, completed in 1883, was the first bridge of this kind in which steel was used for the cables, suspenders, stiffening trusses and floor system.

The Williamsburg Bridge, 1 600 ft. span, completed in 1904, is the latest of the long-span suspension bridges.

The proposed Manhattan Bridge, across the East River, New York City, 1 470 ft. span, is also of this type, a novel feature being the hinged steel towers.

Suspension bridges with braced cables in place of auxiliary stiffening trusses, the cables consisting of forged eye-bars, have also been successfully used for long-span bridges. The Point Bridge, over the Monongahela River, at Pittsburg, of 800 ft. span, completed in 1876, is a bridge of this kind.

In locations where the appearance of the structure is one of the most important considerations, this type eventually may take the place of the cantilever for long-span railroad bridges.

Metal Arches.

Metal arches are particularly suitable for long spans in certain places. The arch combines the advantages of a graceful appearance with facility of erection without false work.

Most of the earlier ones were constructed of cast iron, an important example of which is the Chestnut Street Bridge, in Philadelphia, completed in 1863.

The first important steel arch was the St. Louis Bridge, over the Mississippi River, completed in 1874, consisting of three spans, the middle one, of 515 ft., being the largest.

The highway bridge across the Mississippi River, at Minneapolis, having two spans of 456 ft. each, was completed in 1888.

The Washington Bridge, across the Harlem River, New York City, finished in 1889, consists of two spans, each of 510 ft.

A number of arches of various types followed, the most noted of which are the two across the Niagara River, one of which, of 550 ft. span, carries the tracks of the Grand Trunk Railway and a highway, replacing the Roebling suspension bridge. It was constructed in 1897. The other replaced the Niagara Falls and Clifton Suspension Bridge in 1898. It has a span of 840 ft., and is the largest arch of any type in the world.

Cantilever Bridges.

Cantilever bridges are generally suitable for long spans only; where the length required is too great for a simple truss, or where it becomes necessary to erect without temporary supports, and the conditions are not favorable for an arch. It, perhaps more than any other kind, has been erected in places where simple trusses would have been more appropriate, and freaks of this kind may be seen in various places.

As the cantilever bridge is not as economical as a simple truss, except for spans of great length, and as simple trusses in many cases can be erected on the cantilever principle, the simple truss is generally preferable to the pure cantilever type. The Atbara Bridge, and the bridge recently erected over the Ohio River on the line of the Baltimore and Ohio Railroad at Benwood, are examples of this kind of construction.

The first cantilever railroad bridge was the Kentucky River Bridge, of three spans, each of 375 ft., completed in 1877, on the line of the Cincinnati and Southern Railway.

Many other cantilever bridges were built thereafter, such as:

The Niagara River Cantilever Bridge, on the line of the Michigan Central Railroad, 470 ft. span, completed in 1883. The Frazer River Bridge, on the Canadian Pacific Railway, in 1884, 315 ft. span, and the St. John's River Bridge, in 1885, 477 ft. span. The Poughkeepsie Bridge, across the Hudson River, including a span of 523 ft., finished in 1889. The Colorado River Bridge, at Red Rock, Colo., 660 ft. span, completed in 1890, and the Memphis Bridge, with a span of 790 ft., completed in 1892.

The latest and most conspicuous cantilever bridges constructed are those over the Monongahela River, on the line of the Wabash Railroad, at Pittsburg, 812 ft. span, and at Mingo Junction, 700 ft. span, completed in 1904; also the Thebes Bridge, across the Mississippi, 671 ft. span, completed in 1905. The bridge over the East River at Blackwell's Island, now under construction, has spans of 984 and 1 182 ft., respectively.

The longest cantilever bridge is that across the St. Lawrence River, at Quebec, with a span of 1 800 ft., now being built. This will make it the longest cantilever bridge in the world.

At the present rate of development of bridge building, with the materials at our command, we may see even longer spans in the near future.

It is entirely feasible to build simple trusses of 800 ft., arches and cantilevers of 2 000 ft., and wire-cable suspension bridges of 3 000 ft. span.

Monumental Bridges.

Bridges of monumental nature have generally not received the same careful attention in this country as in Europe. There is really not much difference between public buildings and other public works of like importance. While public buildings have been designed for appearance as well as for permanency, public bridges have been notoriously neglected in both respects. Thus in large cities we see many bridges, which are far from being ornamental or sightly in appearance, situated in prominent places and in public parks in the midst of beautiful landscapes.

Such structures, which are in many cases the most prominent objects in a city, should be monumental in character, and designed on æsthetic as well as on engineering principles. Fortunately, in later years, there has been a manifest tendency toward improvement in this direction.

Any experienced bridge engineer is competent to decide upon the proper design of a structure, wherein utility and economy are the only considerations, but not if the structure requires æsthetic treatment, as engineers are generally deficient in æsthetic training. In the design of a structure of monumental character, the engineer should co-operate with a competent architect, who should be a true artist and not merely a decorator.

The best results can only be obtained by a competition, such as is the usual practice with other monumental structures, the jury of selection to be composed of engineers and architects. This method was adopted in the selection of designs for the proposed Memorial Bridge and the Red Creek Bridge at Washington, and has proved very satisfactory.

The Washington Bridge, in New York City, and the Cambridge Bridge, at Boston (nearly completed), are good examples of monumental city bridges.

Erection.

The erection of structures has kept pace with the rapid progress in structural engineering, requiring the institution of new methods and the development of new appliances demanding engineering skill.

The increased weight of members, consequent upon the development of long spans and lofty viaducts and buildings, and the necessity of assembling the members rapidly and economically, often over treacherous streams, while carrying safely, and without interruption, the constant traffic of a railroad, have made the erection one of the most important operations connected with the construction of the various kinds of engineering structures.

The introduction of the traveler was a marked advance in the erection of structures, dispensing with a portion of the temporary supports. Later, the traveler was developed to erect viaducts and bridges of the cantilever type without the use of any temporary supports.

Until recent years the question of erection was left to the skill and judgment of practical men who had gained their knowledge and had become experts by long experience.

The field operations in connection with the erection of some of the structures of recent years were of such magnitude and complicated character that it required a high order of engineering skill and experience to carry them out to a successful completion.

Erection of bridges and other structures has manifestly advanced from being mere skilled labor, executed by men having only practical training, to be an important branch of engineering.

Nickel Steel.

Nickel steel has of late received special attention, and has been investigated by engineers, in relation to its usefulness as a structural material. For many years, metallurgists have experimented on the effect of the addition of special metals to steel with a view of increasing the ultimate strength and elastic limit of the steel without proportionately decreasing its ductility. So far, as a special structural steel, nickel steel is the only one which has proved satisfactory.

Nickel steels of varying carbon and nickel have been successfully used during the last fifteen years for marine and stationary engine shafting, locomotive axles, piston rods and crank pins, and a wide variety of forgings and castings for parts of machinery. Its application for the manufacture of armor plate, since 1890, is well known. It has recently been adopted, especially in this country, for gun forgings. It has been proposed for structural work before, but is now actually used for bridge construction in the eye-bars for the Blackwell's Island Cantilever Bridge across the East River, New York City, and may take an important place as a structural material for long-span bridges.

Concrete Construction.

Concrete construction has been in use for many years, but is used more extensively now than formerly for foundations, piers and abutments, as well as for bridges. Marked progress was made in concrete construction when the methods of reinforcing concrete

with steel were introduced. Concrete arches reinforced with steel ribs or bars, properly designed and constructed, have proved satisfactory for highway as well as railroad bridges, and are gradually superseding those of masonry construction.

Reinforced concrete is now used successfully in the construction of floors of bridges and buildings. It has also proved satisfactory for fire-proofing and as a protection, to the steelwork of bridges over railroad tracks, against the corroding influence of the gases from locomotives, and will probably take a permanent place in structural work in the future.

Contracting.

Contracting has undergone a marked development to keep pace with the advances made in bridge building and structural engineering. Deep and difficult foundations for bridge piers and high buildings are more frequently required now than in former years, and while the engineer furnishes the design, it devolves upon the contractor to devise ways and means for doing the work. Many of the problems the contractor has to solve require engineering talent of the highest order, and some of the contractors' engineers are men of exceptional ability and of high standing in the profession.

From the foregoing short review, it is evident that this country has been an important factor in the development of the art of bridge building in modern times.

In timber bridges, America has excelled all other countries. Many famous bridges were built during the period when timber was the principal material of construction.

Bridges such as the "Colossus" Bridge, over the Schuylkill at Fairmount, Philadelphia, the Cascade Bridge and the Portage Bridge, on the Erie Railroad, and many other structures, which were masterpieces of work in timber, are considered at the present day marvels of genius and skill in the art of carpentry.

America is the country where the first design and model of a cast-iron arch, which may be considered the prototype of the modern steel arch, was made in 1786, and where was built, in 1796, the first

suspension bridge with iron chains and suspended roadway, and also the first wire-cable suspension bridge, in 1816.

The first bridge in the construction of which steel was successfully used, namely, the famous St. Louis Arch Bridge, was built in America, and America has the largest number of long-span steel trusses in the world. It has the largest steel arch in existence, namely, the one across the Niagara River, with a span of 840 ft., also the longest suspension bridge, the Williamsburg Bridge, across the East River, with a span of 1 600 ft.

American engineers are now constructing the largest cantilever bridge, over the St. Lawrence at Quebec, which, as previously stated, will have a span of 1 800 ft., and will be the longest span bridge in the world.

ERRATA.

Transactions, Vol. LIV.

Page 244, 4th line: For $\frac{7}{8}$-in., read $\frac{5}{8}$-in.
Page 247, 3d line: For \$7.1229, read \$7.0624.
Page 250, 2d line: For \$54.59, read \$24.59.
Page 250, 3d line: For 9.3248, read 9.3169.
Page 254, 23d line: For 0.586, read 0.578.
Page 254, 24th line: For 300.3, read 264.5.
Page 254, 25th line: For 8.338, read 8.330.
Page 254, 3d line from bottom: For 300.3, read 264.5.
Page 254, 2d line from bottom: For leading, read loading.
Page 259, 16th line: For 0.06597, read 0.06610.
Page 259, 19th line: For 19.51, read 19.01.

The above errata refer to Paper No. 994, "The Water-Works of Porterville, California," by Philip E. Harroun, M. Am. Soc. C. E., and should be inserted in *Transactions,* Vol. LIV, page 235.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 994.

THE WATER-WORKS OF PORTERVILLE, CALIFORNIA.*

By Philip E. Harroun, M. Am. Soc. C. E.

With Discussion by
Messrs. D. C. Henny, H. F. Dunham, G. W. Tillson,
William Mayo Venable, Horace J. Howe,
G. L. Christian and Philip E. Harroun.

The engineer finds available much descriptive matter and many data as to the cost of systems of water-works of magnitude, but for those systems supplying small towns and cities little is to be had. For this reason it is believed that a description of the Porterville Water-Works, together with some notes on their operation, will prove of interest to the Profession, not only in the details of design and construction, but especially in the detailed items of cost. These latter items are especially interesting in showing the unit cost of construction and station expense, under such conditions, and the large proportional general and organization charge against the plant.

For such small systems it is usually difficult to find reputable contractors, experienced in this class of work, who are willing to execute the work as a whole, and if an attempt is made to let the various details by contract, the difficulties are greatly increased. In either case the contractor has to meet the same general charges as

* Presented at the meeting of February 15th, 1905.

those which would apply to the construction of a system of much greater magnitude, and, in addition, feels justified in asking a net percentage of profit inversely proportional to the cost of the system. Under such circumstances, the engineer frequently finds it advantageous to undertake the construction of the system by company work, especially where the character of the structures proposed requires care in their execution, and, consequently, a careful supervision. These considerations, particularly the latter, controlled in the decision to construct the Porterville system by company work.

The plans were prepared by Arthur L. Adams, M. Am. Soc. C. E., and the plant was built complete by the writer.

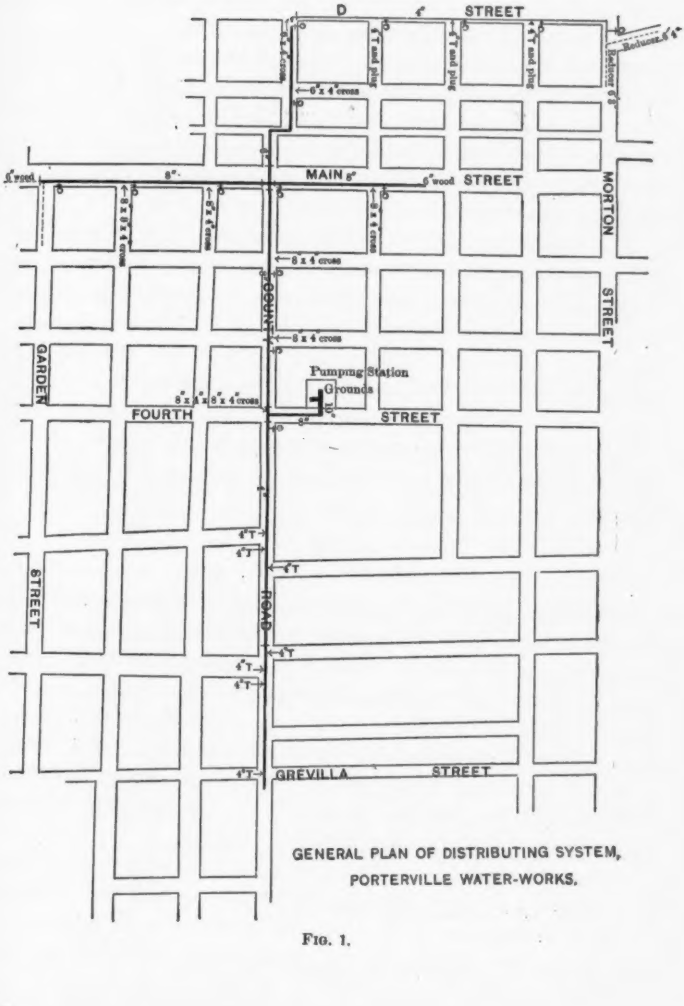
The Old System.

Porterville is situated in the orange belt at the southeastern edge of the San Joaquin Valley, California, and has an estimated population of from 1 800 to 2 000. The water supply was drawn from a bored and cased 12-in. well, 196 ft. deep, passing through from 10 to 12 ft. of surface drift, thence through seamed or laminated clay separated by thin water-bearing gravel strata, to the source of supply, a gravel stratum some 10 ft. thick.

The pump was a 6-in. compound centrifugal, placed horizontally at the bottom of a 6 by 8-ft. timber-lined shaft, 25 ft. deep, and was direct-connected to a 30-h. p., 200-volt, 2-phase motor running at a speed of 850 rev. per min. on a vertical shaft 20 ft. above the pump, the pump motor and shaft being held within a light, four-post, structural-steel tower.

Current was supplied by the Mt. Whitney Power Company at $50 per horse-power per annum for continuous use, this rate being based on the maximum amount of power consumed. The pump was operated continuously, and delivered 360 gal. per min. against a dynamic head of 153 ft. Measurement of current by the Mt. Whitney Power Company showed a consumption of 38 h. p., giving a combined efficiency of about 35% for the plant.

The system in use was pumping direct, and to storage and equalizing tanks. These tanks, of which there were two of 30 000 gal. each, were of 3-in. timber staves, and were supported on a low



GENERAL PLAN OF DISTRIBUTING SYSTEM,
PORTERVILLE WATER-WORKS.

Fig. 1.

timber tower upon a knoll 4 800 ft. east of the town and the pumping plant.

The distribution system consisted of 6 499 ft. of 6-in. riveted steel pipe, No. 16 gauge; 1 359 ft. of 6-in., 721 ft. of 4-in. and 4 403 ft. of 3-in. spiral-wound, wooden stave pipe; 2 140 ft. of 3-in. and 9 842 ft. of 2-in. standard screw pipe. All services, of which there were 215, were of ¾-in. screw pipe. Fire hydrants fitted for 2½-in. hose connections were distributed throughout the business and principal residence sections, and in many instances were taken from 2-in. street mains. The efficiency of the fire service can better be imagined than described.

The riveted steel pipe was second-hand when laid in 1889, and became so badly corroded that in 1892 it was cased in from 4 to 6 in. of sand concrete. The standard screw pipe has been in service since 1889, and is in fair condition, considering its life, but is badly corroded in places. The spiral-wound, wooden stave pipe was laid during the spring of 1902, and the steel-wire winding shows corrosion where its asphaltum coating has been injured. Services were so badly corroded that in many cases the metal had disappeared and the water was carried by a rust-cemented sand shell 3 or 4 in. in diameter.

The New System.

The design for the new system proceeded on the assumption that sufficient funds could be expended for the construction of such works as were necessary to give, as a permanent investment, the most economic, and therefore the most profitable, results, regardless of first cost, it being assumed that the works should be designed with a view of affording a degree of fire protection suited to the needs of a town of from 2 000 to 8 000 people, direct from the hydrants, with the possibility of suitable increase should the growth of Porterville in the future require it. (Fig. 1.)

This policy, however, could not be carried out in full, and, after construction was begun, curtailment of funds available for the plant caused a modification of the plans, the most noticeable being a reduction in the capacity of the elevated tank from 100 000 to 75 000 gal.

It was decided to continue the system of pumping direct and to storage, and to locate the elevated steel tank on the grounds at the pumping station. The old tanks were to be abandoned entirely, as well as 2 600 ft. of the old 6-in. main supplying these tanks and adjoining them, there being no service connections within this section.

Fire service was to be pumpage direct into the mains at 100 lb. pressure, the elevated tank being cut off from the system by a gate at the pumping station.

Gas engines, operating with crude oil, were selected for power, instead of electricity or steam. In the case of electricity, the price asked by the Mt. Whitney Power Company, $50 per horse-power per annum, actually amounted to more than $100 per horse-power, on the basis of the maximum rate used. The resulting annual saving in operating expenses by the use of gas engines was estimated to be about $700, and the saving over the cost of steam power to be nearly as great.

The plant as designed consists of:

1.—A second well, adjacent to the first;

2.—A concrete-lined pump pit about the wells, oval in shape, 18 by 22 ft., on minor and major axes, and 25 ft. deep;

3.—An engine-house covered with galvanized corrugated iron for the housing of the engine plant;

4.—Pumping machinery in duplicate, consisting of two 9 by 12-in. triplex single-acting power pumps, each having a capacity of 500 gal. per min. at 100 lb. pressure, set on the bottom of the pump pit and belt-connected to two 32-h. p. gas engines; both engines and pumps to be operated coincidently only in case of fire;

5.—A concrete-lined reservoir of 100 000 gal. capacity adjoining the pumping plant, to supply one of the pumps in case of fire;

6.—A concrete-lined fuel-oil storage tank of 7 000 gal. capacity;

7.—An elevated steel tower and tank of 75 000 gal. capacity;

8.—A system of cast-iron street mains, ranging in size

from 8 to 4 in., the principal connections about the pumping station and elevated tank being 10 in. in size in anticipation of future need.*

Plates XIX and XX and Figs. 1, 2, 3 and 4 show the general arrangement of the various structures as well as their details.

Construction.

To appreciate the local conditions affecting construction, it should be stated that Porterville is essentially a farming and fruit-raising community. There are no machine shops in the town, nor any facilities for handling pipe work larger than a 2-in. screw. The tools owned by the water-works consisted only of some half dozen picks and shovels. A "farmer's" blacksmith shop constituted the entire resource for any ironwork. All fittings and supplies, of whatever character, and screw-pipe work larger than 3-in., were obtained in San Francisco, 250 miles distant, and sent by freight. A cracked flange or a missing sleeve meant a delay of from eight to twelve days.

Common labor was scarce and unusually poor as a class. Skilled labor, with the exception of carpenters, was not to be had. The general wage paid was $2 for common labor, $3 to $4.37½ for carpenters, $4 for a foreman on pipe work, $3 per day to $90 and $100 per month for a foreman on other work, all rates being on the basis of a 10-hour day.

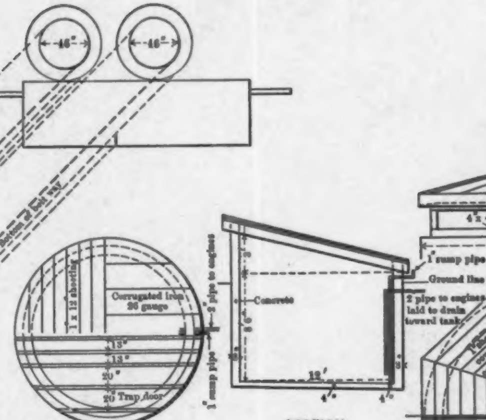
The New Well.—No interest attaches to the drilling of the new well. The formation has been previously indicated. The well was carried to a depth of 216 ft. and cased with single-riveted, steel, slip-joint, double casing, the inner casing of No. 14 gauge and the outer of No. 16 gauge, in 2-ft. lengths. It had been put down and perforated by the Water Company prior to the writer's assuming charge. Its cost is shown in Table 1.

Pump Pit.—In constructing the pump pit it was necessary to preserve without disturbance the centrifugal pump and motor upon which the town depended for its supply. Suspension rods were attached to the upper posts of the steel tower, and by these the pump,

* From Report of Arthur L. Adams, M. Am. Soc. C. E.

PLATE XIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 994.
HARROUN ON
PORTERVILLE WATER-WORKS.

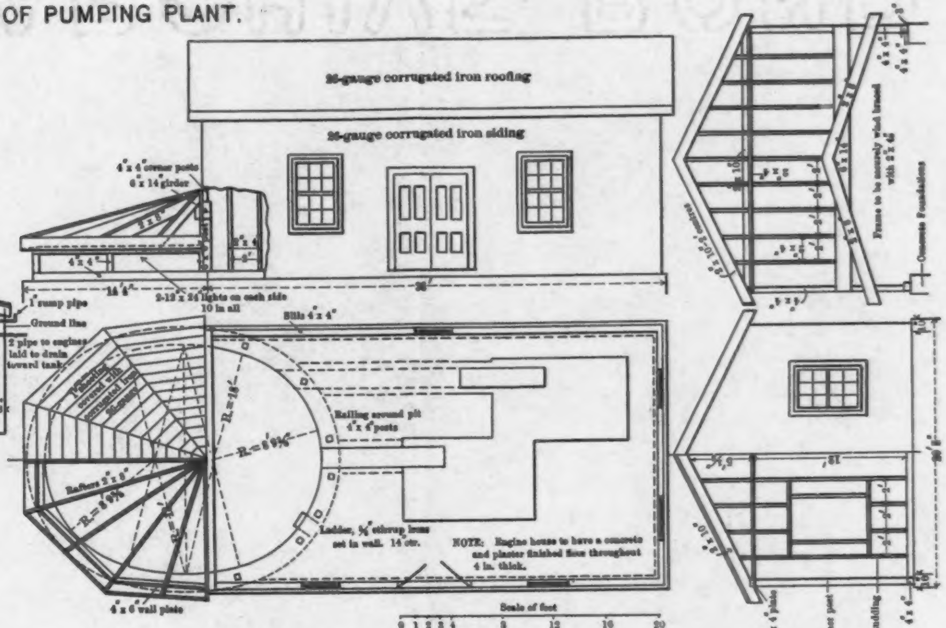
DETAILS OF PUMPING PLANT.

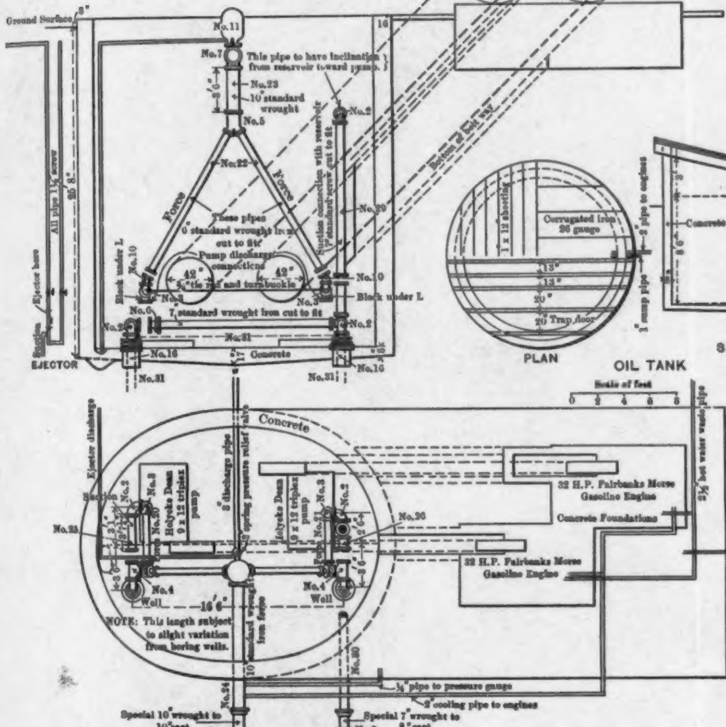


PLAN

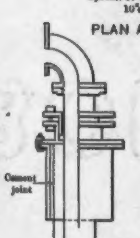
SECTION

OIL TANK





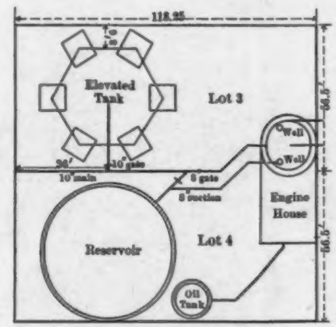
PLAN AND ELEVATION OF PUMP AND ENGINE CONNECTIONS.



CONNECTION OF WELLS AND SUCTION.

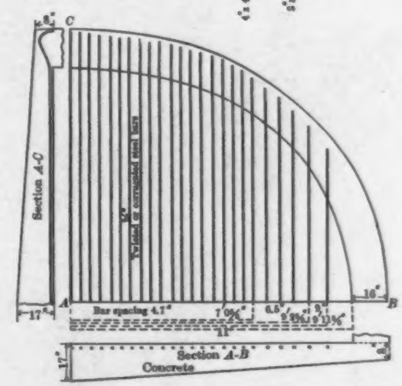
Scale of inches

FLANGES



GENERAL ARRANGEMENT OF PLANT.

Scale of feet



PLAN AND SECTION, BOTTOM OF PUMP PIT.

Scale of feet

motor and piping were hung to timber girders which were supported by blocks outside of the pit excavation.

TABLE 1.—COST OF WELL.

Drilling and driving casing,	200 ft.	at $2.00	....	$400.00
" " " "	16 "	" 2.25	....	36.00
Casing	216 "	" 1.03	....	222.48
Perforating casing				20.00
Testing well				22.50
Incidentals				16.57
Cost of well				$717.55

Excavation and Timbering.—The excavation was wholly by pick and shovel. To hoist the excavated material from the pit, a boom derrick was rigged. It handled two swing-bottom dump-boxes, each having a capacity of ⅓ cu. yd. One box was filled in the pit while the other was being hoisted by a two-mule team, and swung over a waiting wagon and dumped.

In sinking, the upper 7 or 8 ft. encountered consisted of adobe and river silt. Below this there were from 4 to 5 ft. of coarse gravel carrying a heavy volume of surface water. Underlying this water-bearing gravel there was clay to the bottom of the excavation. This clay was very hard to pick and move, and was filled with innumerable seams carrying water under pressure. Although the new well-casing had been perforated throughout this clay and surface-water section, and the city pump while in operation carried the water-table some 40 ft. below ground, yet the pressure of the water within the clay stratum was not relieved. The sides of the pit were covered with spouting streams, and the clay from which these emerged sloughed off badly, while the bottom of the pit was a series of small geysers. On account of this sloughing and caving, timbering became necessary from the surface down.

In timbering, ribs were constructed, 4 by 12-in. in section, oval in shape, 20 ft. 8 in. by 24 ft. 8 in. along the minor and major axes, so as to conform to the outside circumference of the pit concrete.

These ribs were braced radially by timbers ranging from 4 by 6 in. to 4 by 12 in. Outside these ribs 2 by 6-in. sheeting was driven vertically, the driving keeping pace with the sinking of the pit. The vertical spacing of the ribs was adjusted in accordance with the lateral pressure encountered, and the entire system was braced so as to prevent buckling.

During the excavation of the pit no special provision was made for handling the surface water. It had been reaching the city pump through the open 12-in. well at the bottom of the old shaft, and, during the sinking of the pit, it was disposed of in the same way. A slight turbidity in the city water was the only noticeable result.

The distributed cost of the pit and shaft excavation and timbering is given in Table 2.

TABLE 2.—Distributed Cost, Pit and Shaft Excavation, and Timbering.

Segregation.	Time.	Rate.	Total amount.	Cost per cubic yard. (458.9 cu. yd.)	
Excavation:					
Pick and shovel..............	1 238 hr.	20 cents.	$257.05	$0.5663	$0.8236
" " "	31½ "	30 "			
Miscellaneous labor..........	30 "	20 "	7.50	0.0165	
" "	5 "	30 "			
Team hoist...................	130 "	5 "	6.50	0.0143	
Tools and blacksmithing......	....	....	65.32	0.1439	
Foreman......................	....	....	37.50	0.0826	
Timbering:					
Carpenter....................	72 hr.	37½ cents.	53.55	0.1179	$0.6658
"	88½ "	30 "			
Carpenter's helper...........	154½ "	20 "	30.90	0.0681	
Lumber (7 251 ft.)...........	....	....	161.39	0.3556	
Miscellaneous materials......	....	....	18.88	0.0416	
Foreman......................	....	....	37.50	0.0826	
Cost of excavation and timbering............			$676.04	$1.[illegible]94	$1.4894

The cost given in Table 2 covers the depositing of the excavated material in wagons at the edge of the pit, from which point it was removed by other parties without cost to the Company. The small charge for team hoisting is due to the use of the Company's team at cost of feed.

STEEL TOWER AND TANK FOR PORTERVILLE WATER-WORKS.

Concrete.—Before beginning to place concrete, special provision was made for taking care of the surface water. Over the pit bottom was laid a series of open-joint, tile drains leading to a sump and controlled by a gate at the sump. A 5-in. centrifugal pump, belt-connected to a 20-h. p. motor owned by the Company, was installed to handle this water. The bottom of the pit was then covered with 6 in. of gravel, giving the water free access to the drains, and over this was spread cement sacking. Upon the bottom thus prepared was placed the concrete.

Concrete, for the pit bottom and walls, and for the belt shafts, to a point above the ground-water level, 10 ft. below the surface, was mixed in the proportion of 1 part of Portland cement, 2 parts of sand and 4 parts of river gravel. Above this depth, and for all other structures, the proportions were 1:3:5.

The cement used was an American Portland brand, costing $2.25 per bbl. in San Francisco and $3.26 per bbl. laid down on the work. Sand of good quality was secured from a slough ¾ mile distant, at a cost of 41.7 cents per cu. yd. River gravel was procured from bars in the Tule River, from 1½ to 2½ miles distant.

These bars contained from 80% to 85% of sand and from 15% to 20% of gravel, thus necessitating screening. The nearest bar yielded 315 cu. yd., at a cost of $1.42 per cu. yd., and the upper bar, 126 cu. yd., at a cost of $2.70 per cu. yd.

The concrete was mixed by hand, on a platform at the edge of the pit, in batches of approximately ½ cu. yd., and lowered into the pit by horse hoist. In general, the gang consisted of seven men on the ground, two preparing the batches and five mixing, watering, filling hoist boxes and operating the hoist. Three or four men worked in the pit, placing, tamping and finishing.

In mixing, the gravel was first laid down and leveled off in a 3-in. layer. The sand was then placed on this and leveled off, and a layer of cement was placed on top. Each batch was then mixed twice dry and twice wet, water being supplied from a hose. The concrete was mixed quite wet, so as to flush readily under the tamping iron.

No forms were necessary in placing the bottom concrete. For the pit walls, oval ribs of 2 by 12-in. timber were built up of such dimensions as to allow 1-in. boards or lagging, placed vertically

against the outer edge of the rib, to bring the ribs to the true section of the finished concrete. These ribs were spaced 2 ft. apart, vertically, and 1 by 6 by 24-in. lagging was used in forming the staves.

The plans called for ⅞-in. steel bars in the bottom concrete, with a spacing of from 4.7 to 9 in. These bars could not be obtained in the California market in less than 90 days, and, in order to avoid this delay, ¼ by 2-in. "medium" steel bars were substituted, thus using 50% more steel than called for.

All interior surfaces of the pit and shafts were covered with a cement-plaster finish, in the proportion of one part of Portland cement to 1½ parts of sand, and troweled to a smooth finish.

The writer has now to record one of the most disagreeable features of the work. Tests of the Portland cement, for fineness, soundness and tensile strength, were made before placing the order for the lot required in this work. No chemical analysis was made. The tests were conducted under the rules of the American Society of Civil Engineers, and showed great uniformity in grinding, no checking or blowing, and a tensile strength somewhat greater than the average. In fineness, the average of six samples passing the 10 000-mesh sieve was 96.9 per cent. In tensile strength, the average of six samples of neat cement at two days was 447 lb.; seven days, 692 lb.; and 28 days, 899 lb. Initial set took place in from 1 hr. 45 min. to 2 hr., and final set in 5 hr. 30 min. No tests for fineness or tensile strength were possible at Porterville, but, on receipt of the cement, samples were taken from 10-bbl. lots, and pats were made which were observed for set and soundness. No defects in the cement were to be observed through the action of these pats, at the time of making or at any subsequent time.

The cement was received at the work in one lot of 1 800 sacks (450 bbl), and within one week was being used in the pit concrete. Its action in the concrete was good, initial and final set taking place as usual, its entire behavior being apparently normal.

About 8 weeks after the concrete was placed it began to disintegrate, breaking down completely into its original constituents, with no cohesion whatever to be observed anywhere within the mass. This action did not take place throughout the entire body of concrete, but was confined to the pit bottom and lower portion of

the walls below the ground-water line, or about 16 ft. above the bottom of the pit, and occurred here only in pockets, other portions remaining perfectly sound. The pit and belt shafts contained 118 cu. yd. of concrete, and, of this, 28 cu. yd. had to be replaced on account of this disintegration.

The cause of this disintegration is believed to have been due to the cement being insufficiently burned and possibly insufficiently ground, and also to excess of lime. It has been learned that the superintendent of the cement works, who is said to be thoroughly proficient, was unavoidably absent from the works for some two weeks, and that at least a portion of the cement for the Porterville shipment was turned out during that time.

TABLE 3.—Cost of Concrete for Pit and Belt Shaft.

Items.	Quantities.	Rate.	Total amount.	Cost per cubic yard. (146 cu. yd.)	
Labor:					
Mixing and lowering	508¼ hr.	20 cents.	$101.65	$0.7966	$1.7897
Team lowering	293 "	5 "	14.65		
Placing	67 "	30 "	20.10	0.4295	
"	213 "	20 "	42.60		
Tamping	102 "	30 "	30.60	0.4835	
"	200 "	20 "	40.00		
Laying steel	20 "	30 "	6.00	0.0801	
" "	28½ "	20 "	5.70		
Forms:					
Carpenter	116 "	37½ "	43.50	0.7464	0.7464
"	96½ "	30 "	28.95		
" helpers	182½ "	20 "	36.50		
Miscellaneous:					
Labor	194¼ "	....	47.42	0.3247	1.2474
Sump pump	119¾ "	....	26.73	0.1830	
Foreman	....	....	108.00	0.7397	
Cost of labor			$552.40	$3.7885	$3.7835
Materials:					
Sand	....	....	$25.14	$0.1722	$7.2257
Gravel	....	....	269.80	1.8479	
Cement	678 sacks.	81.6 cents	553.25	3.7893	
Steel	2 440 lb.	....	113.95	0.7804	
Tools and blacksmithing	....	....	39.42	0.2700	
Lumber for forms	2 155 ft.	....	53.42	0.3659	
Cost of materials			$1 054.98	$7.2257	$7.2257
Cost of labor			552.40	....	3.7835
Total cost of concrete			$1 607.38	....	$11.009

The cost of the concrete for the pit and the belt shaft is given in Table 3.

The breaking down of the concrete, due to defective cement, was not only expensive on account of the increased yardage, but was disastrous in affecting the cost of the plaster finish. During the time elapsing between the first finish of the work and its replacement, the surface-water drainage behind the pit lagging had become clogged, causing a static pressure against the concrete, according to its depth below the ground-water line. The concrete was found to be pervious, as all concrete is, and the percolation through the new concrete destroyed the new plaster coat, wherever placed, before it had time to set. Weep-holes through the work proved to be of no value, and each section broken by the replacement of the concrete was taken in hand separately.

Beginning about the circumference of such a section, the plaster was placed over a small area, and the blue flame of an ordinary plumber's gasolene torch was blown directly on the finish. The green finish set at once, adhering well to the concrete behind it. In this way the weeping area was gradually reduced, and, when brought to somewhat less than 18 in. in diameter, a plank form, shaped to the walls and surrounded by a rubber ring gasket, the interior being filled with cement plaster, was placed against the weeping area and held tight by a screw-jack until the enclosed plaster had well set. This method was expensive, as plaster finish goes, but was very effective, the total seepage through the pit walls, bottom, and belt shafts, being only 12 gal.* per hour on the completion of the work, and most of this was due to the water following along the ladder irons in the pit wall. The first cost of the plaster finish amounted to \$3.07 per square (100 sq. ft.) for labor and \$3.98 for materials; total, \$7.05 per square. The repair cost, in the aggregate, \$145.45.

Engine-House.—The engine-house consists of framing covered with corrugated galvanized iron, the dimensions and details being shown on Plate XIX. The superstructure rests on concrete foundations. The floor and engine foundations are of concrete covered with a smooth cement-plaster finish. Gutters, built in the concrete

* This was handled by a hydraulic ejector or jet pump, operating on service pressure, at practically no expense.

floor, carry all engine piping. The cost of the completed structure was as follows:

Concrete, 41 cu. yd. at $7.1229..............		$289.56
Plaster finish, including floating of engines...		44.22
Superstructure, Labor................	$140.35	
Materials	322.92	463.27
Total cost of engine-house..............		$797.05

Pumping Machinery.—The pumping machinery consists of two 32-h. p. gas engines, belt-connected to two 9 by 12-in. triplex, single-acting, power pumps. These two engines are operated at the same time only in case of fire.

Engines.—The engines are of the four-cycle type, with water-cooled cylinders and fly-wheel governors. Their normal speed is 200 rev. per min., which is capable of hand regulation to 40% below or 10% above normal. Ignition is effected either by a sparking dynamo, belt-connected to the engine shaft; by a 12-cell sal-ammoniac carbon-zinc battery; or by a hot tube, as may be advisable.

Cooling water is taken from the city mains, and, after leaving the engine cylinder, is discharged by gravity into an irrigating ditch at the back of the engine-house. The engines operate either on gasolene, distillate, or on Coalinga crude oil, although the intention was to operate them wholly on the crude oil. The engine is fitted with two suction pumps, one for gasolene or distillate, and the other for crude oil. By these pumps the fuel is drawn to the engine and then by force pump carried into the cup at the receiving valve port. In operating, the engine is started on gasolene or distillate, upon which it may be run entirely. If crude oil is to be used, the exhaust from the engine is deflected, by a damper, through a converter. After this converter has become thoroughly heated, the crude oil is admitted, vaporized and carried to the engine as a gas, the change from gasolene or distillate to crude oil being made gradually by closing off the supply of the former and admitting the latter. Any surplus fuel drawn to the engine by the pumps is returned to the supply tanks by gravity lines. From time to time,

the residuum from the crude oil or distillate passed through the converter is drawn off, while hot, from a collecting reservoir at the bottom of the converter.

These engines are operated with all grades of gasolene and distillate above those of 45° gravity, taking the charge cold. The converter is used when those of lower gravity are used.

The engines are connected to the pumps by 7-in., eight-ply, double-stitched, rubber belting. No loose pulley is used, the load, in starting, being thrown off by by-passes on the pumps. Belt tighteners, throwing in and out by hand-gear, are provided for use, if necessary, at maximum load.

The engines were contracted for at a price, f. o. b., Porterville, the contract also providing for hauling them to the plant, setting them on the foundations and connecting the piping.

Pumps.—The pumps are 9 by 12-in. triplex, single-acting, with outside-packed plungers and machine-cut gears. Each has a capacity of 500 gal. per min. at a pressure of 100 lb. per sq. in. They were made by the Dean Steam Pump Company, of Holyoke, Mass. The gearing is in the ratio of 8 to 1. Each pump is by-passed by connecting the suction, immediately in front of the pump, to the discharge, by a 4-in. vertical riser and an angle valve, a 10-in. check-valve being inserted in the main between the engine-house and the elevated tank to prevent back pressure when the by-passes are open.

The pumps were contracted for at a price, f. o. b., Porterville, the contract also providing for hauling them to the pit and setting them on the foundation bolts in the bottom of the pit.

Table 4 gives the cost of the pumping machinery, erected and complete.

Oil Tank and Setting Gasolene Tanks.—The oil tank, as designed, consists of an 8-in. circular shell of concrete, 12 ft. inside diameter,and with 4 in. of concrete on the bottom, the whole being finished inside with cement plaster in the proportion of 1 part of cement to 1½ parts of sand. The roof, of galvanized, corrugated iron over 1-in. sheeting, rests upon stringers set in the concrete. The tank is set 8 ft. in the ground; and suction, feed and gravity return

pipes are carried to the engines. A 4-in. inlet pipe is provided for filling, and there is a 1½-in. hand sump pump to draw off accumulating water. In excavating, the material encountered was a heavy adobe. The cost of the structure is given in Table 5.

TABLE 4.—Cost of Pumping Machinery, Erected, Complete.

Engines.			
Two 32-h. p. engines (contract price)		$2 860.00	
Haul, placing on foundations and connecting piping (contract price)		90.00	
Belt tighteners, two	$76.25		
" " framing and placing	22.55		
		98.80	
Fittings, foundation bolts, tubes, etc.		47.59	
Labor, lining up, adjusting, etc., 124 hr. at 30 cents		37.35	
Belting		141.48	
Miscellaneous materials		10.79	
Total cost of engines			$3 286.01
Pumps and Pump Pit Fittings.			
Two 9 by 12-in. single-acting triplex pumps (contract price)		$2 816.00	
Haul and placing on foundations		170.00	
Foundation bolts, tubes and setting same		41.65	
Special castings		372.12	
Pipe, flanges and bolts		248.01	
Valves		160.68	
Fittings, gaskets, miscellaneous and blacksmithing		133.97	
Labor, connecting up		99.60	
Ejector, pipe fittings and connecting up		38.30	
Cost of pumps and pump pit fittings			$4 080.33
Total cost of pumping plant			$7 366.34

TABLE 5.—COST OF OIL TANK, ETC.

Excavating (pick and shovel) 44 cu. yd.	at $0.5588	$54.59
Concrete, 13 cu. yd.	" 9.3248	121.12
Plaster finish, 6.3 squares	" 3.0064	18.94
Roofing		51.96
Sump pump and fittings		5.18
Setting two 65-gal. gasolene tanks		13.20
		$234.99

Two weeks after the concrete was placed, the plaster finish was put on, the concrete appearing wholly normal. Fifteen days after plastering and thirty days after concreting, a carload, 6 500 gal., of crude Coalinga oil was received, and, after a thorough examination of the tank, when everything was found to be apparently in good condition, this oil was run in. The oil was transferred from the car to the tank by a small tank wagon, holding from 300 to 350 gal., and was run into the tank through a 1½-in. rubber hose. Before the oil was of sufficient depth to form a cushion, it fell directly on the concrete bottom, about 9 ft. below the inlet. Owing to the small capacity of the wagon and interference caused by construction work about the plant, it was a little more than two days before all the oil was transferred to the tank.

No evidence of seepage through the tank or of anything abnormal was noticed during this time. During the evening of the day on which the last oil had been run into the tank, the city water began to taste of oil. This was attributed to the lubricating oil used in the stuffing-box of the centrifugal pump having worked down into the pump, as this had frequently occurred before.

On the following morning the city water was undrinkable, and pure oil ran from the taps when first opened. Examination of the plant showed that the ground-water contained oil, which entered the cased well below the concrete bottom of the pit in which the city pump operated. The oil in the tank was at once transferred to barrels, and the water from the city pump was cut off from the mains and wasted in an adjoining ditch, while a temporary supply

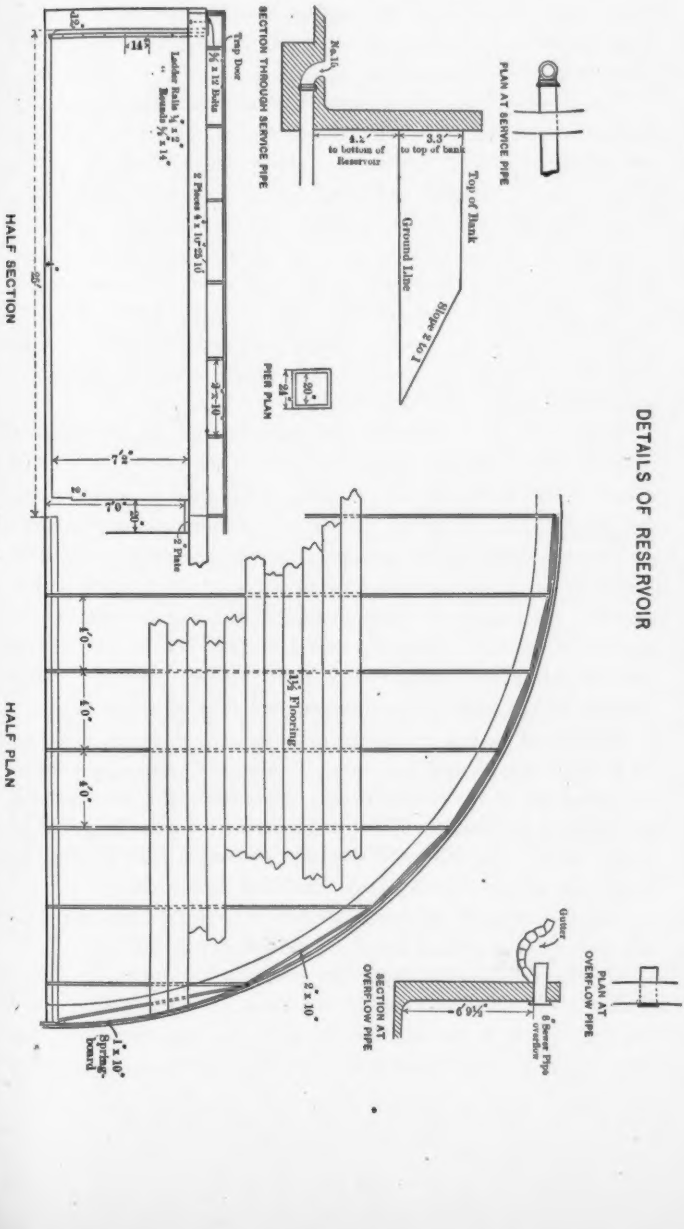

DETAILS OF RESERVOIR
PLAN AT SERVICE PIPE
Top of Bank
Ground Line
Slope 2 to 1
4.2′ to bottom of Reservoir
3.3′ to top of bank
No. 15
SECTION THROUGH SERVICE PIPE
PIER PLAN
20"
24"
Trap Door
7′2"
7′0"
20"
2" Plate
2"
14"
12"
25′
HALF SECTION
HALF PLAN
4′0"
4′0"
4′0"
1½ Flooring
2" x 10"
1" x 10" Spring-board
PLAN AT OVERFLOW PIPE
SECTION AT OVERFLOW PIPE
Gutter
8" Sewer Pipe overflow
6′9½"

FIG. 2.

for the town was obtained by tapping a canal from the Tule River which passed near the storage tanks east of the town. This gave a pressure of about 15 lb. per sq. in.

The oil tank, upon being emptied, showed nothing abnormal except under the spot where the oil was run in, and there, where the oil stream fell on the bottom, the plaster finish, covering an area about 1 ft. in diameter, had disappeared. This erosion must have taken place when the first oil had been run into the tank, and before sufficient depth had been obtained to cushion the falling stream. It was attributed to the defective cement to which reference has been made. Although this eroded area unquestionably added materially to the rate of seepage through the tank walls and bottom, this seepage took place over the entire wetted area, as was proved by excavating along the outside of the tank concrete.

That the lost oil through the tank did not appear sooner is probably due to the impervious character of the adobe soil in which the tank was built, the oil requiring some time to work its way through this soil into the gravel stratum carrying the ground-water.

Concrete tanks, for the storage of fuel oils having a gravity ranging from 15° to 20° on the Beaumé scale (0.967 to 0.936 specific gravity), are frequently and successfully used. In this instance the gravity of the oil is variously stated at from 30° to 35° Beaumé (corresponding to from 0.880 to 0.850 specific gravity). Where may the dividing line between success and failure be placed?

In making repairs, a riveted-steel tank, with soldered seams, was built up of No. 20 steel. It was 1 in. less in diameter than the inside diameter of the concrete tank. This steel tank was then lowered within the concrete shell upon a bottom of grout prepared for it, and, after being filled with water to prevent it from floating, the ½-in. space between the two tanks was filled with grout.

The total seepage loss of oil through the concrete tank was 2 133 gal. and the total cost of the direct repairs was $264.50.

Reservoir.—The details of the reservoir are shown in Fig. 2. Owing to the small size and to the shape of the excavation, *i. e.*, circular and 52 ft. in diameter, the plow and slip scraper could be used only to a slight extent and to but little advantage. The excava-

FOUNDATION FOR TOWER AND TANK AT PORTERVILLE, CALIFORNIA

CAPACITY 75 000 GAL.

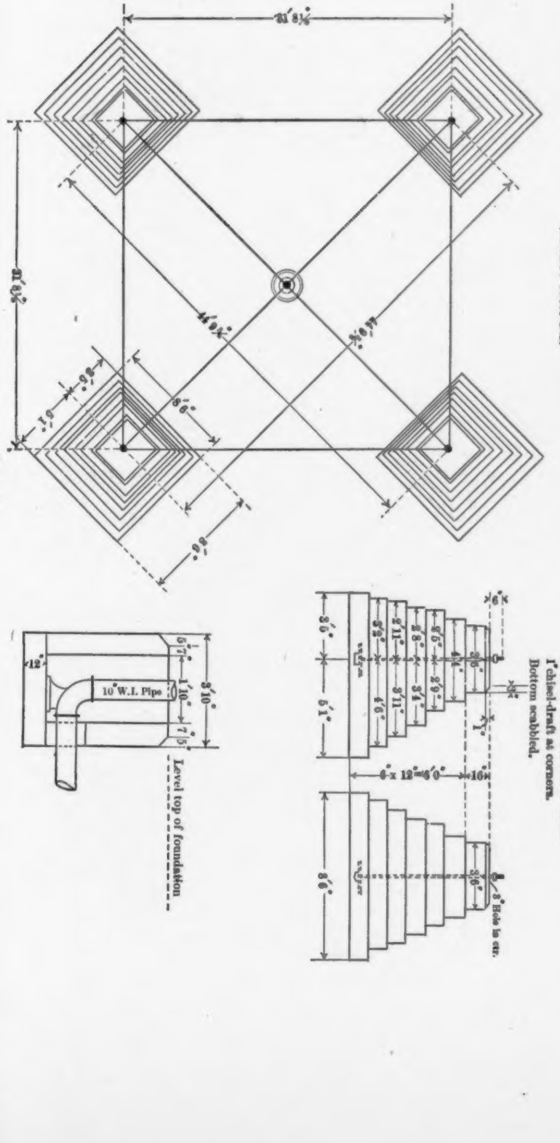


FIG. 8.

tion was principally pick and shovel work, and the material encountered was a heavy adobe. The excavation was carried to a depth of 4.2 ft. and banked up around the outside, forming a berm with a 4-ft. crest and with slopes of 2 to 1 on the outside. This berm and the natural soil were afterward trimmed vertical on the reservoir side, the trimmed surface forming the outside of the concrete wall. The bottom was also carefully trimmed to a close grade.

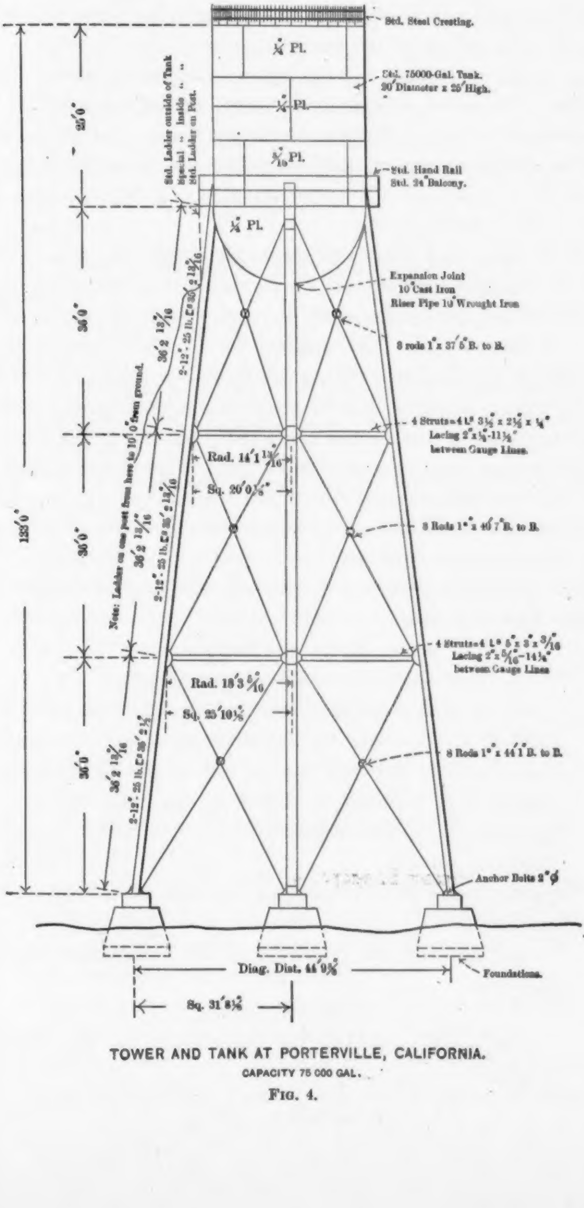
For forms for the circular wall, triangles of 2 by 4-in. and 2 by 6-in. scantlings were made, the 2 by 6-in. piece being 8 ft. long and placed vertically, one 2 by 4-in. piece at right angles and extending on the ground toward the center of the reservoir, while the second was used as a tie-brace. They were held in place by wooden pins driven into the bottom. Against the faces of these triangles were sprung 1 by 12-in. by 16-ft. planks, the concrete being deposited between these planks and the trimmed earth. Four triangles were used for each 16-ft. section of wall. The method of mixing and the proportions of the concrete used have been already described.

The roof was flat, and was of 1 by 12-in. undressed lumber on joists 4 ft. apart, between centers. The joists were supported by a diametrical girder resting on a central pier and the reservoir walls. The cost of the structure is given in Table 6.

TABLE 6.—Cost of Reservoir.

Excavation	330.4 cu. yd. at 0.586	$191.08
Haul	300.3 " " " 0.204	53.98
Concrete	75 " " " 8.338	624.74
Plaster finish	35.1 squares " 2.918	102.45
Roof	4 000 ft., B. M. at 45.49	181.96
Trimming outer slopes		18.70
Total cost		$1 172.91

The cost of excavation includes loading 300.3 cu. yd. of material, and hauling it a distance of about ¾ mile. The cost of this loading, hauling and disposal amounted to $53.98, and it was sold for $107.71,



TOWER AND TANK AT PORTERVILLE, CALIFORNIA.
CAPACITY 75 000 GAL.

FIG. 4.

leaving a profit of \$53.73. For the concrete, the labor cost \$3.026 and the material \$5.312 per cu. yd.

In plaster finishing, no attempt was made to remove trowel marks. The labor charge for vertical surfaces was \$1.116 per square; for horizontal surfaces, \$0.574 per square; and the materials cost \$1.859 per square, in each case. The roof labor cost \$12.43 per 1 000 ft., B. M., and the material \$33.06, of which lumber cost \$29.635 per 1 000 ft., B. M.

Steel Tower and Tank.—As previously stated, the original intention was to erect a six-post steel tower and a tank having a capacity of 100 000 gal., but curtailment of funds caused the final selection of a 75 000-gal. tank. The contract for this tank was let to the Chicago Bridge and Iron Works, on their own plans and standard specifications, for furnishing the material and erecting it on foundations prepared by the Porterville Water Company. The company also furnished the riser and overflow pipe. Figs. 3 and 4 show the dimensions of the principal details, and Plate XX is a photograph of the tower and tank. All steel was inspected, and also all shopwork, at a cost of 75 cents per ton.

The specifications provided that the structure be proportioned for the following loads:

1.—The weight of the structure;

2.—The weight of the water in the tank;

3.—A wind pressure of not less than 30 lb. per sq. ft. over one-half of the diametrical plane of the tank, and a uniform load of 200 lb. for each vertical foot of the tower; the wind forces being assumed as acting in any direction, and the members to be proportioned for that direction giving the maximum stresses.

The unit stresses for proportioning members are as follows:

COMPRESSION:

For members not exceeding 90 radii of gyration between supports:

14 000 lb. per sq. in.

For length exceeding the above limit, use the following formula:

$$P = 20\,300 - 70\frac{L}{R}$$

P = Allowed stress per square inch;

L = Length between supports;

R = Least radius or gyration, in inches.

No mainpost to exceed 100 radii of gyration in length.

No other strut to exceed 150 radii of gyration, or such length that the fiber stress due to bending from its own weight exceeds 4 000 lb. per sq. in.

TENSION:

10 000 lb. per sq. in., net section.

SHEAR:

7 500 lb. per sq. in.

BEARING:

15 000 lb. per sq. in. on rivets; 20 000 lb. per sq. in. on pins; 400 lb. per sq. in. on capstones; and 125 lb. per sq. in. on masonry.

For wind stresses, the above unit stresses may be increased 25 per cent.

All metal in the structure was required to be soft steel. All steel comprising the tank plates and principal parts of the main posts was to be made by the open-hearth process; other steel was to be either open-hearth or Bessemer.

Rivet steel was to show an ultimate strength of from 48 000 to 58 000 lb. per sq. in.; an elastic limit not less than one-half the ultimate strength; an elongation of 26%; and a bending test of 180° flat, upon itself, without fracture on the outside of the bent portion. The soft steel was to be the same as the rivet steel, except that the ultimate strength was to be from 52 000 to 62 000 lb. per sq. in. and the elongation 25 per cent.

In open-hearth steel made by the acid process, the phosphorous limit was to be 0.08%; in that made by the basic process, 0.04 per cent. All material received one shop coat of graphite paint, and was given a second coat after erection.

The tank is provided with a wooden roof of dressed redwood on joists, the latter resting upon the reinforcing angles so placed as to bring the top of the roof flush with the upper edge of the tank. The roof is provided with a scuttle hole over a ladder extending to

the bottom of the tank on the inside. A 5-in. overflow pipe passes through the top plate immediately below the upper reinforcing angle. Outside of the tank, it was reduced to 3 in.

The riser pipe is 10-in., standard, flanged screw-pipe, and the gaskets are of ⅛-in. lead.

The foundation bolts are round, 2 in. in diameter and 7 ft. 4 in. long. The capstones are of granite, 3 ft. 6 in. square and 16 in. high.

The foundations are of concrete, in the proportions of 1 part of cement, 3 parts of sand and 5 parts of gravel. Each block is battered, so that the line of thrust passes through the center of gravity of each element. In excavating for the foundations, the character of the soil was such as to necessitate going much deeper than contemplated. Consequently, the yardage of both the excavation and the concrete was greatly increased. The excavated material from the pits, except that used for back-filling, was loaded into wagons, hauled ¾ mile and deposited, at a cost of 20.4 cents per cu. yd. It was sold at a clear profit of $15.30.

TABLE 7.—Cost of Concrete in Tank Foundations.

Segregation.	Quantities.	Rate.	Total amount.	Cost per cubic yard. (104.7 cu. yd.)	
Labor:					
Wheeling sand and gravel	234½ hr.	20 cents.	$46.90	$0.4479	$1.2740
Mixing	190½ "	20 "	38.10	0.3638	
Placing	123 "	20 "	24.60	0.2349	
Tamping	119 "	20 "	23.80	0.2274	
Forms:					
Carpenter	64 "	30 "	19.20	0.1835	0.1835
Miscellaneous and foreman	....	....	44.48	0.4248	0.4248
			$197.08	$1.8823	$1.8823
Materials:					
Sand	....	....	14.18	$0.1354	$5.8592
Cement	387 sacks.	81.6 cents.	315.79	3.0101	
Gravel	....	....	276.96	2.6452	
Form lumber (old)	....	....	....	....	
Tools	....	....	7.19	0.0685	
Cost of materials			$614.12	$5.8592	$5.8592
Labor and forms			197.08	1.8823	1.8823
Total cost of concrete			$811.20	$7.7415	$7.7415

The concrete in the tank foundations was placed at less cost than that in any other part of the work, except in the engine-house. Its distributed cost is given in Table 7, in comparison with the distributed cost of the pit concrete. It will be observed that the cost of gravel in this case was $2.70, while that in the pit concrete was only $1.42 per cu. yd.

The cost of the completed structure is given in Table 8.

TABLE 8.—Complete Cost of Elevated Tank.

Excavation (pick and shovel), 156.9 cu. yd.	at $0.6484 ...	$101.74
" back-fill .... 52.2 " "	" 0.1226 ...	6.40
" loading, hauling and delivering 104.7 cu. yd., excavated material ¾ mile distant	" 0.204 ...	21.35
Concrete, 104.7 cu. yd....................	" 7.7415 ...	810.53
Capstones, 65 cu. ft. in place.............		231.55
Tower and tank, 78 532 lb. steel, erected....	" 0.06597...	5 191.00
" " " riser pipe, 10-in. standard screw-pipe, 102 ft. 2 in................................		269.23
Tower and tank overflow and miscellaneous................		19.51
Total cost of entire structure.......................		$6 650.81

Tank Ratios.—It is interesting to compare the ratios between the weights of the tanks, of the tower, and of each entire structure, with the weight of the water in each tank, and observe the economy in material in proportion to the capacity. These ratios, also, may be of value for preliminary estimates of the weights of tanks and towers, and consequently their cost, where the heights and capacities approximate those given and where the loading and unit stresses are the same.

Cast-Iron Pipe, Services and Connections.—The streets of Porterville are not paved, but have been crowned, covered with from 2 to 4 in. of mixed clay and gravel, and rolled hard. In trenching for the pipe, a 4-horse subsoil plow was used to break through this crust. The material below was a heavy adobe, except for 900 ft. of

4-in., 300 ft. of 6-in. and 600 ft. of 8-in. pipe, where sand was encountered. The width of the trenches was 18 in. in all cases, and the average depth 3 ft. 4 in. In back-filling, a "go devil" was used wherever practicable to throw the material into the trench, where it was consolidated by water from the street hydrants.

TABLE 9.—Weights and Ratios.

100 000-gal. tank, 22 by 28 ft. Spherical bottom. Height of tower, 95 ft. Height above foundations, 133 ft.

Weight of tank	39 600 lb.
" " tower	49 773 "
Total	89 373 "
Ratio: weight of tank to weight of water	1: 21.04
" " " tower " " " "	1: 16.74
" " " entire structure to weight of water	1: 9.32

75 000-gal. tank, 20 by 25 ft. Spherical bottom. Height of tower, 98 ft. Height above foundations, 133 ft.

Weight of tank	34 260 lb.
" " tower	44 272 "
Total	78 532 "
Ratio: weight of tank to weight of water	1: 18.20
" " " tower " " " "	1: 14.12
" " " entire structure to weight of water	1: 7.96

The fire-hydrants are of the Greenburg type, San Francisco standard. They are provided with two 2½-in. outlets and are fitted for 4-in. and 6-in. connections to street mains. After setting, they were well backed with boulders about their bases.

Valve boxes are placed over all valves, short pieces of cast-iron pipe, placed bell down, being used. All service connections are ¾-in. Corporation cocks and flange couplings are of brass, taper-threaded, of the Mueller lead-flange type, with extra heavy, lead-pipe, gooseneck. The wages paid on this pipe work were as follows: foreman $4.00, caulkers $2.50, yarners $2.25, and laborers $2.00 per day of 10 hours. The distributed cost of this work is given in Tables 10 to 15, and Table 16 gives a summary of the cost of the complete plant.

TABLE 10.—Cost of Laying 4-inch Pipe.

Laying length of pipe....................	2 820 ft.
" " " specials	26 "
Total laying length....................	2 846 "

Segregation.	Quantities.	Rate.	Total amount.	Cost per foot.	
Labor.					
Trenching............	991 hr.	20 cents	$198.20	$0.0696	
" team........	16 "	5 "	0.80	0.0004	
Bell-holes	220½ "	20 "	44.10	0.0154	
Laying...............	137 "	20 "	27.40	0.0096	
Yarning..............	68 "	22¼ "	15.19	0.0054	
Pouring..............	54 "	20 "	10.80	0.0038	
Caulking.............	87 "	25 "	21.75	0.0078	
Back-filling	160½ "	20 "	32.10	0.0013	
" team.....	76 "	5 "	3.80	0.0014	
Distribution..........	22.443 tons.	60 "	13.47	0.0047	
Miscellaneous.........	56	....	12.89	0.0040	
Foreman	....	....	47.52	0.0166	
Timekeeper	....	....	6.27	0.0023	
Cost of labor..............................			$434 29	$0.1529	$0.1529
Materials.					
Pipe, 2 820 ft..........	32 150 lb.	$44.40 per ton.	$1 312.50	$0.4612	
Specials	4 462 "	0.03¼ per lb.	145.01	0.0509	
Valves................	9	9.40 each.	84.60	0.0297	
Hydrants	5	28.60 "	143.00	0.0502	
Lead	2 010 lb.	0.05328 per lb.	107.09	0.0376	
Yarn	105 "	0.0541 "	5.68	0.0020	
Tools................	....	....	42.49	0.0149	
Miscellaneous.........	....	....	26.19	0.0092	
Cost of materials....			$1 866.56	$0.6557	$0.6557
Cost of labor..............................			434.29	0.1529	0.1529
Total cost..............................			$2 300.85	....	$0.8086

TABLE 11.—Cost of Laying 6-inch Pipe.

Laying length of pipe........................816 ft.
" " " specials .... 22 ft.

Total laying length........................838 ft.

Segregation.	Quantities.	Rate.	Total amount.	Cost per foot.	
Labor.					
Trenching.............	315¼ hr.	20 cents.	$63.10	$0.0753	
" team........	5 "	5 "	0.25	0.0003	
Bell-holes.............	70 "	20 "	14.00	0.0169	
Laying...............	56 "	20 "	11.20	0.0133	
Yarning...............	19 "	22½ "	4.27	0.0051	
Pouring...............	27 "	20 "	5.40	0.0064	
Caulking..............	33 "	25 "	8.25	0.0098	
Back-filling...........	51 "	20 "	10.20	0.0122	
" team......	24 "	5 "	1.20	0.0014	
Miscellaneous.........	18 "	....	4.07	0.0049	
Distribution	17.194 tons.	60 "	10.32	0.0123	
Foreman..............	....	....	15.12	0.0180	
Timekeeper..........	....	....	2.00	0.0024	
Cost of labor......................................			$149.38	$0.1783	$0.1783
Materials.					
Pipe, 216 ft............	26 244 lb.	$43.40 per ton.	$569.49	$0.6788	
Specials................	1 420 lb.	0.03¼ per lb.	46.14	0.0550	
Valves	10	15.65 each.	156.50	0.1866	
Hydrants..............	9	29.85 "	268.65	0.3203	
Lead..................	804 lb.	0.05328 per lb.	43.27	0.0516	
Yarn..................	42 lb.	0.0541 "	2.27	0.0027	
Tools..................	....	....	13.52	0.0161	
General...............	....	....	8.33	0.0099	
Cost of materials..............................			$1 108.17	$1.3210	$1.3210
Cost of labor....................................			149.38	0.1783	0.1783
Total cost..			$1 257.55	....	$1.4993

Operation.—At the time of writing, the plant has been in operation a little more than three months, and some notes on its performance may be of interest.

The population served by the plant is about 1 200, and the plant was designed on the basis of an average daily consumption of 125 gal. per capita,* which rate it was expected would be reached by ordinary restrictive measures, and inspection on the part of the

*The excessive consumption under the old system had been pointed out in the first reports, and attention called to the necessity for reducing it.

management. This rate of consumption represents an average pumpage of 5 hr. per day, which would allow one man to attend to the plant while running and also do all the outside work, making taps, repairs, collections, etc., thus reducing the cost of attendance and superintendence to the salary of this one man.

Table 17 covers the pumpage and cost of operation of the plant from March 22d, when it was turned over to the operating department, to June 30th.

TABLE 12.—Cost of Laying 8-inch Pipe.

Laying length of pipe.......................2 512 ft.
" " " specials .... 46 "

Total ....2 558 ft.

Segregation.	Quantities.	Rate.	Total amount.	Cost per foot.	
Labor:					
Trenching	901 hr.	20 cents	$180.20	$0.0706	
" team	15 "	5 "	0.75	0.0003	
Bell-holes	200½ "	20 "	40.10	0.0158	
Laying	202 "	20 "	40.40	0.0159	
Yarning	66 "	22½ "	14.85	0.0059	
Pouring	76 "	20 "	15.25	0.0061	
Caulking	129 "	25 "	31.75	0.0125	
Back-filling	146 "	20 "	29.20	0.0115	
" team	69 "	5 "	3.45	0.0013	
Miscellaneous	51 "	....	11.63	0.0037	
Distribution	65.862 tons	60 "	39.52	0.0155	
Foreman	....	....	43.20	0.0165	
Timekeeper	....	....	5.70	0.0022	
Cost of labor			$456.00	$0.1778	$0.1778
Materials:					
Pipe, 2 512 ft	115 225 lb.	$43.40 per ton	$2 500.39	$0.9774	
Specials	4 056 lb.	0.03¼ " lb.	131.83	0.0515	
Valves	5	24.00	120.00	0.0469	
Lead	3 618 lb.	0.05328 per lb.	194.72	0.0761	
Yarn	189 lb.	0.0541 " "	10.22	0.0039	
Tools	....	....	38.62	0.0151	
Miscellaneous	....	....	23.81	0.0093	
Cost of materials			$3 019.59	$1.1802	$1.1802
Cost of labor			456.00	0.1778	0.1778
Total cost			$3 475.59	....	$1.3580

TABLE 13.—Cost of Laying 10-In. Pipe.

Laying length of pipe........................124 ft.
" " " specials .... 14 "

Total ....138 "

Segregation.	Quantities.	Rate.	Total amount.	Cost per foot.	
LABOR:					
Trenching	120 hr.	20 cents	$24.00	$0.1738	
Bell-holes	10 "	20 "	2.00	0.0145	
Laying	15 "	20 "	3.00	0.0217	
Yarning	1½ "	20 "	0.30	0.0022	
Pouring	1½ "	20 "	0.30	0.0021	
Caulking	10 "	20 "	2.00	0.0145	
Back-filling	41 "	20 "	8.20	0.0595	
Miscellaneous	10¼ "	20 "	2.05	0.0148	
Distribution	4.533 tons	60 "	2.72	0.0197	
Foreman	....	....	2.16	0.0155	
Timekeeper	....	....	0.30	0.0021	
Cost of labor			$47.03	$0.3404	$0.3404
MATERIALS:					
Pipe, 124 ft.	7 500 lb.	$43.40 per ton	$162.75	$1.1790	
Specials	603 "	0.03¼ per lb. and 0.04½ " "	24.59	0.1781	
Valves	1	34.60 each	34.60	0.2507	
Lead	268 lb.	0.05328 per lb.	14.42	0.1045	
Yarn	14 "	0.0541 " "	0.76	0.0054	
Tools	....	....	1.93	0.0148	
Miscellaneous	....	....	1.19	0.0086	
Cost of materials			$240.24	$1.7411	$1.7411
Cost of labor			47.03		0.3404
Total cost			$287.27	....	$2.0815

TABLE 14.—Cost of Making 78 ¾-inch Service Connections.

Segregation.	Quantities.	Rate.	Total amount.	Cost per connection made.	
LABOR:					
Trenching	239 hr.	20 cents.	$47.80	$0.6128	
Tapping and making	195½ "	40 "	78.20	1.0026	
" " helper	112½ "	20 "	22.50	0.2885	
Back-filling	80½ "	20 "	16.10	0.2064	
Cost of labor			$164.60	$2.1103	$2.1103
MATERIALS:					
Goosenecks and cocks	78	2.48 each.	$193.64	$2.4800	
Fittings	....	....	31.31	0.4000	
Tools	....	....	68.33	0.8760	
Tapping machine	....	....	80.75	1.0353	
Cost of materials			$373.73	$4.7913	$4.7913
Cost of labor			164.60	....	2.1103
Total cost			$538.33	....	$6.9016

TABLE 15.—COST OF MAKING CONNECTIONS BETWEEN OLD AND NEW PIPE SYSTEMS.

Six 2-in., three 3-in., and three 6-in. (lead).

Labor.	Hours.	General rate.	Total amount
Trenching	153	20 cents.	$30.60
Making	30	40 "	12.00
" helpers	104	20 "	20.80
Back-filling	45½	20 "	9.10
Miscellaneous	45½		13.65
Materials, fittings, etc.			13.22
Total			$99.37

TABLE 16—SUMMARY OF COMPLETE COST OF CONSTRUCTION.

	Dr.	Cr.
To well	$717.55	
" pump pit	2 588.30	
" engine-house	797.05	
" pumping machines	7 366.34	
" oil and gasoline tanks	499.59	
" reservoir	1 172.91	
" elevated steel tank	6 650.81	
" distribution system	7 958.96	
" wrecking city pump	24.84	
" maintenance, city pump, during construction	393.25	
" grading and improving grounds	43.14	
" general	2 851.93	
By cash received for sale of earth		$144.35
" " " " " " old form lumber, etc.		73.29
" appraised value of stock on hand and tools		1 171.19
" balance: net cost of system		29 675.86
	$31 064.67	$31 064.67

The cost of pumping, in Table 17, is much less than the average for plants of this size, and compares very favorably with the cost in plants of a capacity many times as great.

The pumping record represents a probable pumpage of about 121 000 000 gal. per annum, or an average daily consumption of 280 gal. per capita. The minimum daily consumption occurred in March, and was at the rate of 150 gal. per capita; the maximum daily consumption (to date) occurred in June, and was at the rate of 465 gal. per capita. For such a town, with no manufacturing interests whatever, it is believed that these figures are without a parallel. This excessive consumption is due to several causes. The

system is entirely unmetered, and some water is sold for the irrigation of lawns, etc., not in connection with domestic use, at a flat rate of 50 cents per month per lot of 50 ft. front. Prior to the installation of the new plant, the use of water by consumers was only limited by the capacity of the old plant. Since the new plant has been running, restrictive measures have been attempted, especially in the use of water for irrigation, and, while the consumption has been greatly reduced, the rate is still unprecedented. As an instance of the excessive use of water for irrigation, it may be stated that a meter was purchased by the Water Company and placed on the service pipe of a consumer using water for irrigation only. This consumer in eight days used 15 000 gal., or at the rate of 56 250 gal. per month. In the following month, after being informed that he would be required to pay for water as used, his entire consumption was only 6 150 gal.

Taking the cost of pumping at 13½ cents per million gallons per foot raised, and all other charges, interest, depreciation, etc., as 12 cents, the cost of the water supplied in this instance was $2.37, and the revenue, $0.50.

Fuel and Duty.—The calorific value of the fuel oils used in the plant is uncertain, but the writer believes it may be taken at an average of 19 600 B. t. u. for the crude Coalinga oil. This oil ranges between 30° and 35° on the Beaumé scale, which, at 32°, would give it a weight of 7.24 lb. per gal. At the time the plant was designed, it was proposed to use this fuel, which was then selling at 75 cents per barrel (42 gal.) at the wells. During the construction of the plant, the wells were acquired by the Standard Oil Company, and the price was immediately raised to $1.50 per barrel. At this rate, one car load was secured for the plant, after which the Standard Oil Company refused to sell any more at any price. This forced the Water Company to resort to the distillates or to install converters adapted to the heavier oils.

The distillate used is purchased in the open market from independent refineries, and is a by-product, as most of these refineries pass the crude oils through the stills to secure the asphaltum base. Nothing is known as to the calorific value of this distillate. Its

gravity gives it a weight of 7.24 lb. per gal., as in the case of the Coalinga crude. It has been generally supposed that these low-gravity distillates had a heat value of from 5 to 10% in excess of the heavy crude oils, but in this case the engine performance shows that the distillate used has a calorific value of only 81.5% of the Coalinga crude oil, or 15 974 B. t. u.

The writer believes that there is but little possibility of material error in this particular case, but, unfortunately, has been unable to make a personal test. The result is arrived at from the actual performance of the plant extending over the last fifteen days of June.

TABLE 17.—PUMPAGE, FUEL CONSUMPTION, STATION EXPENSE, ETC., FROM MARCH 22D TO JUNE 30TH, 1904.

	Mch. 22-31.	April.	May.	June.
Total pumpage in gallons	1 660 000	7 887 500	12 678 001	18 831 080
" time pumped	55 hr. 20 m.	253 hr. 26 m.	458 hr. 6 m.	521 hr. 47 m.
Average consumption per tap (230) per day, in gallons	732	1 158	1 778	2 005
Average consumption per capita supplied per day, in gallons (consumers estimated at 1 200)	138	219	341	384
Average pressure on gauge, in pounds	50½	51⅓	52½	52¼
Average vacuum, in inches	11½	11⅓	11	10½
Dynamic head, in feet	160	162	164	163
Fuel consumption, in gallons:				
[a] Crude Coalinga oil	196	1 055	1 515	551
[b] Distillate, 55° gravity	112	106	70⅓	92
[c] " 32° "	....	....	....	1 393
Residuum, in gallons:				
From Coalinga	(?)	69	133	46
" 32° gravity, distillate	....	....	....	41
[d] Supplies:				
Engine oil, in gallons	11	13	22	30
" gasolene, in gallons	(?)	5	5	5
Pump oil, in gallons	9	16	25	27
" compounds, in pounds	....	5	8	7
Waste, in pounds	8	10⅓	19	16
Cost:				
Fuel	$19.22	$52.10	$66.37	$84.39
Supplies	10.88	18.40	28.89	33.51
[e] Attendance	32.26	107.50	130.12	180.00
Total station expense on above items	62.36	178.00	225.38	297.90
Cost per million gallons raised 1 ft. high, on above station expense	0.221	0.138	0.108	0.132

[a] Cost at wells, 3.57 cents per gallon. Freight and haul, 0.33 cent. Cost in tank 3.9 cents per gallon.

[b] Cost in tank, 10⅓ cents per gallon.

[c] Cost at refinery, 3.5 cents per gallon; freight and haul, ⅓ cent per gallon; cost in tank, 3.83 cents per gallon.

[d] Cost of engine and pump-lubricating oils, 50 cents per gallon; gasolene, 20 cents per gallon; waste, 11 cents per pound; gear compound, 25 cents per pound.

[e] Full time of Superintendent, at $100 per month, charged to station in March. In April, three-quarters time of Superintendent and full time of First Assistant, at $65 per month, for one-half month. In May, one-half time of Superintendent, full time of First Assistant, and seven days for Second Assistant, at $65 per month. In June, one-half time of Superintendent and full time for First and Second Assistants.

The 55° distillate is what is commonly called engine distillate, and is put on the market especially for use in engines of the gasolene type. Its calorific value is also unknown, but will not fall below that of the Coalinga crude, and will probably run a small percentage in excess. It is used in this plant simply to start the engines, being taken by them "cold." The engines are run on it until the generator has become sufficiently heated to handle the crude oil or heavy distillate.

Taking the performance of the plant through the month of May, the duty developed while operating on the crude Coalinga oil was 10 937 000 ft-lb. per gal. (7.24 lb.) of oil, or 1 510 600 ft-lb. per pound of oil. With bituminous coal at 12 740 heat units, this performance corresponds to a duty of 232 400 000 ft-lb. per 100 lb. of coal.

Taking the heat value of 32° distillate used, as previously indicated, the duty during the last fifteen days in June was, on the same basis, 8 918 000 ft-lb. per gal. (7.24 lb.) of distillate, or 1 232 000 ft-lb. per pound of distillate and 154 500 000 ft-lb. per 100 lb. of coal.

The best performance of the plant, extending over a period of several days, has been 1.43 pints of crude Coalinga oil per horse-power-hour. On the basis of 1 pint per horse-power-hour, brake test at the engine, this represents a combined efficiency of 70% for pump and belting, and the engines at ordinary performance are operating on 1.015 pints of crude Coalinga oil, or 1.146 pints of 32° distillate per horse-power-hour.

The increased cost of pumpage per million gallons per foot raised, from 10.8 cents in May to 13.2 cents in June, is due to the low value of the 32° distillate used.

The Coalinga crude oil is a very clean oil, there being but little accumulation in the generator or deposit in the engine cylinder. On the other hand, the 32° distillate is exceedingly dirty, leaving a gritty deposit within the cylinder, and requiring frequent cleaning and repacking of piston rings.

The writer has had difficulty in securing authentic data as to the fuel value of oils, and submits the following equivalents, obtained from various sources, believing that they may be of value to others.

Crude Bakersfield oil, 14 to $15\frac{1}{2}°$ gravity, contains, per pound, 19 388 B. t. u.

Crude Bakersfield oil, 14 to $15\frac{1}{2}°$ gravity, through refinery, per pound, 20 061 B. t. u.

Low-grade engine distillate, under 50° gravity, 18 000 B. t. u.

For use under steam boilers, the following equivalents may be used:

147.4	gal.	crude	oil	equals	1	long	ton,	white-ash steam coal,
168	"	"	"	"	"	"	"	Beaumont coal,
164	"	"	"	"	"	"	"	Walls End coal,
193	"	"	"	"	"	"	"	Welsh anthracite coal.

In general, for steaming purposes, 2 bbl. of ordinary heavy fuel oil may be taken as equal to 1 cord of good pine wood; and 3.6 bbl. of oil as equal to 1 ton (2 000 lb.) of bituminous coal.

The weights of oil will range from $7\frac{1}{4}$ to $7\frac{3}{4}$ lb. per gal. (42 gal. per bbl.).

Wooden Stave Pipe.—This pipe has been mentioned in reference to the old system, and that it was not wholly replaced with cast-iron pipe was due to lack of funds at the time. Since the new plant has been in operation, this pipe has caused much trouble. It is what is called 150-ft. pipe; that is, it is claimed by the makers to be designed for a static pressure of 150 ft., with a factor of safety of 4 at this pressure. The pipe is spiral-wound, in sections 12 ft. long, and in no part of the Porterville system is it subjected to a static pressure exceeding that for which it is claimed it is designed.

Some forty-one sections, or 492 ft., of this pipe have had to be taken out and replaced within the last three months. The trouble is caused by the variable pressure in the mains. This causes a gradually deeper seating of the winding into the wood, and results ultimately in the opening of the longitudinal seams between the staves. As the winding is spiral, there is no possibility of recinching, and the pipe is worthless.

In closing, the writer wishes to acknowledge his indebtedness to A. L. Adams, M. Am. Soc. C. E., for notes on the condition of the old plant, and to Mr. Charles Byers for his efficient services as Assistant during the latter half of the construction.

DISCUSSION.

Mr. Henny.

D. C. Henny, M. Am. Soc. C. E. (by letter).—The writer has read this paper with great interest, and is impressed with the difficulties which frequently confront an engineer when called upon to design and build small works. Conscientious solution involves the same careful consideration and study as is required for works of great magnitude, yet, if preliminary investigations and tests of material used in construction were carried to the same extent, the engineering expense would reach an objectionably high ratio of total cost.

The author's experience with a portion of the cement used in his works indicates the chances which an engineer in such case is forced to take, and indeed is warranted in taking.

Gasolene Engine vs. Induction Motor.—The reason advanced for the selection of gasolene engines for operating the pumps at Porterville appear to have been purely financial, and it is stated that an estimated annual saving of $700 has probably been effected. The writer would infer from this that conditions at Porterville are quite abnormal. Reducing the author's figures of cost of pumping during June, 1904, presented in Table 17, to the net horse-power unit, they stand as follows:

	Cost per net horse-power hour.
Fuel	0.89 cent.
Supplies	0.35 "
Attendance	1.90 "
Total	3.14 cents.

It will be noted at once that the cost of attendance is high, as is generally unavoidable with small, gasolene-driven plants. Moreover, this item is not subject to much reduction in future, as the plant appears to have been working close to its full capacity for the time actually run.

The great advantage of electric drive, for small plants, lies in the fact that practically no attendance is required. The writer visited Porterville some years ago, and incidentally inspected the old motor-driven centrifugal pumping plant which was at the time running without attendance, and he was then informed that the pump ran practically all the time and required no attention beyond starting and stopping and the occasional filling of the oil reservoirs.

In a system of water-works in California, with which the writer has been connected for several years, three gasolene pumping plants of 15, 25 and 36 h. p., respectively, have been in use, for which the

cost of attendance per net horse-power hour ranged from 1.25 to 3.00 cents, according to conditions and sizes of plant. The same system also contains a 15-h.p. motor-driven pump which has now been in use about two years, and, in regard to which, the superintendent reports that cost of attendance is so small as to be negligible, as the pipeman throws the current on and off when he leaves and returns to the adjacent stable. Mr. Henny.

For purposes of comparison, the writer submits the following estimate of cost of pumping by electric power under the usual conditions in California on a basis of cost of current at the rate of $100 per horse power per year for current actually used, as per meter measurement, assuming 60% efficiency for the motor and pump.

	Cost per net horse-power hour.
Power	1.92 cents.
Supplies	0.10 "
Attendance	0.53 " (variable)
Total	2.55 cents.

For the same amount of work as was done in Porterville during June, the foregoing estimated cost of attendance would amount to $50 per month, which is high.

In first cost, an induction motor is also more economical than a gasolene engine, and, furthermore, it is compact and requires less foundation and housing. Thus, the fixed charges will be relatively smaller, and, likewise, it will show a less depreciation, owing to its simplicity and freedom from wear.

While the conditions in Porterville may have fully justified the selection of gasolene engines, in the majority of cases where electric current is available, at reasonable rates, it is believed that the latter will give better financial results in small plants.

Spiral-Wound Stave-Pipe.—The author's comments regarding the failure of pipe of this class at Porterville are of interest, in view of its growing use, factories having sprung up at half a dozen points throughout the West. The writer manufactured some of this pipe, 4 and 6 in. in diameter, more than 10 years ago, to be used in places where it would be free from serious fluctuation of pressure. The wire was No. 12, galvanized, the pressure was light and the factor of safety of the wire was from 5 to 6. In order to secure a proper bearing of the wire upon the wood, the diameter of the wire must either be small, or the weight of metal must be made excessive, otherwise there is danger from indentation and consequent leakage, especially where fluctuations of pressure occur.

The writer realized that small wire, no matter how well galvanized or coated, cannot be expected to have a long life when in con-

Mr. Henny. tact with alkali soil, and for this reason he abandoned the manufacture of pipe of this class long ago, although that manufactured at the time is believed to be still in use.

As now manufactured, this pipe is wound with $\frac{1}{4}$, $\frac{5}{16}$ and $\frac{3}{8}$-in. wire for sizes less than 12 in. The spacing of the windings seems to be determined generally for the strength of steel alone, the danger from indentation being entirely ignored. The results at Porterville, as recorded by the author, may be considered a confirmation of formulas established long ago.

Mr. Dunham. H. F. Dunham, M. Am. Soc. C. E.—The Society should be grateful for this paper, although its very excellence may be the cause of trouble for some of our members. The author's work was well carried out, then well written out, and now it has been well read out, made real and so like actual experience that every member accustomed to water-works construction may find himself unconsciously including Porterville in his own list of completed plants.

A President of the Technical Society of the Pacific Coast is somewhere quoted to the effect that engineers should conceal their mistakes; but the builder of this water-works speaks from a slightly different position, on the Slope, for he tells all that he has done. No one can ask for more. His work does not show the usual number of errors, if indeed there were any errors. There can never be enough such papers.

Members who have seen a 3-in. shaft neatly welded in an old army forge, and who believe that in nearly every community there is latent mechanical talent, may wonder why the author did not bring to the work a small engine lathe, an 8-in. pipe threader and a portable forge, wherewith to avoid delays, earn more than the cost of the machines, and afford comfort to those who have idle hours when operating the works. But this is too small a matter for review; so also was the application of oil to troubled waters.

Because of the fact that every plant must be designed to meet local conditions, and, further, because the speaker has no knowledge of those conditions, it is evident that any discussion of the general plan must be in the nature of an inquiry, and cannot be regarded as criticism.

It is always interesting and generally desirable to have on record a good sanitary and quantitative analysis of the water to be supplied to a city. It is helpful in many ways. The effect of the water upon pipes may be predicted, to some extent; also its effect upon paint; and it is possible that such analysis might show that no paint would ever be required inside a metal stand-pipe or tank. Then the effect of light upon the water is disclosed by an analysis, and, if necessary, extra care can be taken to cover the stand-pipe or reservoir.

The question of supply, where well water is used, must always be important. Wells are frequently located at considerable distances from each other, and, when they are quite near, it is worth while to note why they are not separated, and how much more water can be obtained from two than from one. The method of perforating well pipes after they are in position is not common in the East, and is not well understood by the speaker. An important object in such work is to exclude sand or silt at the maximum draft of water. A Cook strainer is sometimes effective. Is the pipe perforated in place effective? Mr. Dunham.

It is a good feature in any design to introduce as few parts as possible, if they will give the required service. Evidently, it was deemed necessary at Porterville to have an elevated tank of 75 000 gal. capacity and to store about 100 000 gal. for protection against fires. The cost of the elevated tank and its foundations would not have been greatly exceeded by the cost of a plain stand-pipe, of about the some height and diameter, and its foundations; and such a stand-pipe, connected with by-pass and pressure valve to the pumps, would give a storage capacity more than twice as great as the ground reservoir affords. Or, if the designer had recalled the fact, set forth by Professor Pence, that something unexpected has happened to about 25% of all the stand-pipe or tank structures in the country, and decided to pump direct and use the available funds for the storage of water for fire protection, and possibly for a third pump, a much larger quantity could have been stored and more permanent work secured, with, perhaps, greater safety when fires occur, and no greater expense for operation, since the quantity of water now demanded by consumers must make constant attendance at the station necessary. In fact, such a station, upon which fire protection depends, should never be left without a competent, wide-awake engineer. The speaker has designed and built covered reservoirs of the same inside diameter, 50 ft., but much deeper, doming them over with masonry, without any central or inside support, and covering them with earth. The cost of such permanent work, with a concave bottom to resist upward pressure if the reservoir is quickly emptied, may be said to decrease per 1 000 gal. of water stored with the increase in the size of the reservoir. It may be pertinent to inquire why the Porterville design included a depressed reservoir and an elevated reservoir. A by-pass between them, not mentioned in the description, would enhance the present value of both when the tank is not used for fire protection.

The Porterville map shows the water-works located apparently in the center of the town or city, where presently there may be from 6 000 to 8 000 inhabitants. No contours are shown, and if there were it is doubtful whether they could offer a fair explanation. No

Mr. Dunham reason is given for making that location or for continuing it. Almost every State Board of Health Report carries pages of caution against such locations, for it is always recorded that reasons existed for making an analysis of the cistern water, that too much chlorine was found, that afterward a crack in the cement wall was discovered, and, lastly, that polluted ground-water from somewhere found its way in when the water was low in the cistern. A repetition of all this on a larger scale is apparently invited in Porterville. Adobe soil may have unfamiliar qualities, but oil permeates it. On a flat surface, 4 in. of concrete may be water-tight, but it cannot be expected to resist the upward pressure of a head of even a few inches, if rainfall in that country ever gives such a head when the water is suddenly lowered in the reservoir. The wells may take water at a depth of hundreds of feet, but well pipes are not always new, and new pipes are not always perfect.

Doubtless there are reasons for the design, and it must not be assumed that those reasons are associated in any way with the fact that the greatest sum a city can raise for improvements is too often the least sum which can be found in an engineer's estimate.

The personal equation in cement work, particularly in small undertakings, is so important that it dwarfs nearly everything else. One may have almost any number of excellent tests from cement, and a good quality of cement, sand, gravel and broken stone, but if one fails to secure good foremen there may be very bad work. This has no relation to anything in the paper. It is a relation of experience when the work is not of such magnitude as to warrant ideal conditions. A good foreman should not be under iron-clad rules in such cases. He should use his judgment, reject poor material, make changes in the specified proportions when necessary, and always work to get good and permanent results. In the filtration of water it is expected that workmen will soon be able to determine at a glance whether the effluent contains too many bacteria. It is not unreasonable to expect a foreman to determine whether there is virtue in cement when he is interested in the question and when his work is one continuous decision upon that question.

Mr. Tillson. G. W. Tillson, M. Am. Soc. C. E.—This paper is worthy of special commendation, because it gives, not only the method, but the cost, of construction, in regard to labor and the quantities of material required. While descriptions of methods of construction of many and large works are common, it is difficult to obtain their cost. That, of course, is perfectly natural, because nearly all are built by contract and the contractors keep to themselves the actual cost of the work. Therefore, when a paper gives in detail, as this does, the quantities of work and material used, it should receive special mention.

The author's allusion to the morning when it was possible to draw oil from the water taps calls to mind an occurrence which came under the speaker's observation in Omaha, some years ago, when a gentleman one night went into his parlor to light his gas. When he turned on what he supposed was the gas, he found nothing but water. Although this was rather surprising, it was easily explained, as most things of that kind are when all the facts are known. In a nearby street the 20-in. water main, under quite a heavy pressure, burst, and, as it burst, threw up the earth, and broke the gas main, consequently, the water flowed into the gas main and filled that and also the service pipes, so that when the jet was turned on the gas and water escaped. Mr. Tillson.

In reading the technical part of the paper the speaker's attention was called to the cement and the resulting disintegration of the concrete. The difficulties which an engineer meets in carrying on a work of that kind, where he must take some small chances, are fully appreciated, and yet it would seem that this is a good time to emphasize the necessity of making tests of cement for such work, and also state how those tests should be made. Neat cement tests are of no particular value, unless something is known about the particular brand of cement that is being tested. In order to understand the cement, fully continued tests, both neat and with sand, must be made, so that an approximate ratio can be established between the neat and the sand tests, because any tests of cement without other information are of value only as the conditions which surround the tests conform to the conditions which surround the material as it is actually in the work. Now, cement when used is mixed with sand, so that it is absolutely necessary to establish this ratio without a thorough knowledge of the brand of cement to be used. From the tests given by the author he certainly would be justified in expecting good results in his concrete, as they compare very favorably with the tests of first-class cements used in the vicinity of New York City. The paper does not state that the actual cement used in the work was tested, but that, previous to buying the cement, the brand was tested, and, as there were no facilities for testing the cement while the work was going on, chances had to be taken. It is very difficult to understand why the concrete should have acted as it did. It will be noticed that, although all the concrete that failed or disintegrated was below the water line, all the concrete that was below the water line did not fail or disintegrate. There are some cements, which, if allowed to set dry, will give good results, but if used in water immediately after mixing will disintegrate, but it seems strange that concrete should last eight weeks, and set normally up to that time, and then disintegrate.

Mr. Tillson. A case that came under the speaker's notice a number of years ago showed the necessity of testing cement while it was being used. In a town in the South, on the Mississippi River, in the construction of a sewerage system, it was necessary to build a structure across a bayou which backed up from the Mississippi. One of the division engineers was put in charge of the work and told by the consulting engineer that he would be back in a month, and that at that time he expected to see the structure completed. The engineer went to work promptly, constructed his coffer-dam of the most approved form, and pumped it out. By the time the water was pumped out the bayou was dry, and there was no need of a coffer-dam. He proceeded, however, to construct the piers, of brick and cement. After he had them constructed he decided that the cement was bad, and pulled the piers down. He explained that the cement passed the pat tests, and that he did not see what was the matter. By the time the piers were torn down the month had expired, and the consulting engineer had returned and found the conditions exactly as he had left them, except that the engineer had a four weeks' pay roll to be accounted for.

Mr. Venable. William Mayo Venable, Assoc. M. Am. Soc. C. E.—The speaker would like to know of any places where oil has been stored successfully in concrete tanks. He has been asked about such cases several times, but has not been able to answer the inquiry. Explicit data regarding the imperviousness of well-made concrete to oils of various densities would be very interesting.

Mr. Howe. Horace J. Howe, M. Am. Soc. C. E.—The author describes how a small pumping plant may be operated with the distillate from crude oil, and how the expense of maintenance is reduced thereby to a minimum under the conditions at Porterville. It is a vital question to many owners of country houses whether a much smaller similar plant could not be designed for use, separate and apart from any system of water supply. There is frequently an objection, in the mind of a city person used to home conveniences, to going away for the summer, solely on account of the change of water, both as to quality and quantity.

The speaker is not aware of any motor on the market, either gasolene, kerosene or hot-air, that will raise 500 gal. to the second story of an ordinary country cottage, and do it every day, at as reasonable a cost to the man of small means as the old-fashioned force pump which the speaker used 25 or more years ago.

Mr. Christian. G. L. Christian, Assoc. M. Am. Soc. C. E.—It would seem to the speaker that on all important constructions the cement should be given an accelerated test, in addition to the usual tests for specific gravity, fineness, time of set, tensile strength, etc.

Mr. Christian.

Although the author states that the cement was tested for soundness, he does not state, specifically, in what manner, and it would be interesting to know whether any accelerated tests were made.

Surprise has been expressed in relation to the author's statement that the cement had acted well in the work for eight weeks before showing signs of deterioration. About two years ago the Bureau of Sewers of New York City made some tests of an American Portland cement which acted in a somewhat similar manner.

The cement tested 100% fine, and was made up into thirty-five neat-cement briquettes, which were immersed in clear, fresh water twenty-four hours after making, and kept there until broken, with the results shown in Table 18.

TABLE 18.

Number of briquettes.	Time.	Average tensile strength, in pounds per square inch.	Remarks.
3	24 hours.	180	
3	72 "	690	
6	7 days.	740	
4	28 "	870	
3	3 months.	760	
5	4 "	....	Dissolved in the water.
1	4½ "	....	" " " "
1	5 "	....	" " " "
1	6 "	....	" " " "
2	6 "	660	
1	7½ "	....	" " " "
2	9 "	1—849 1— 54	
3	1 year.	1—848 1—478 1—865	

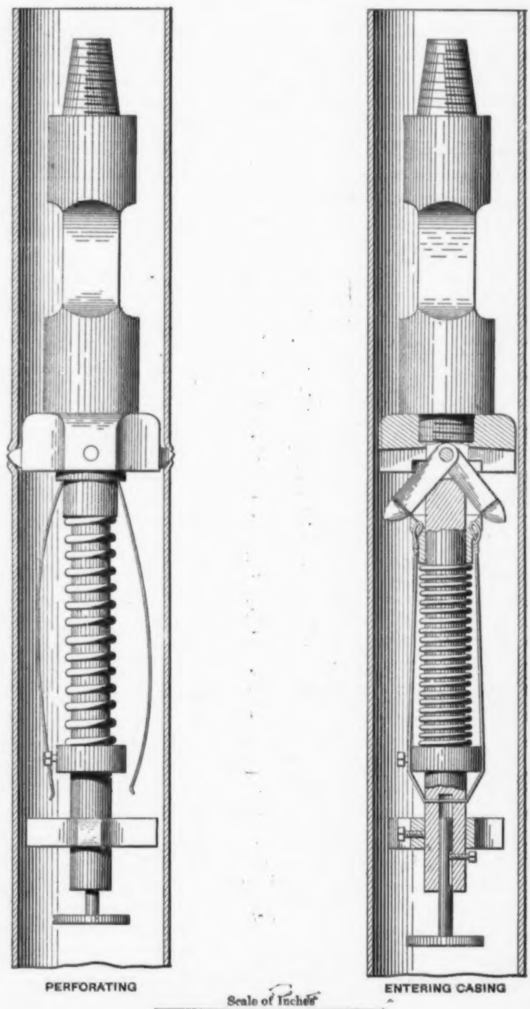
The briquettes first began to show signs of disintegration about three months after making. This cement, unfortunately, was not subjected to an accelerated test, so that it is not known what such a test would have shown.

Mr. Harroun.

Philip E. Harroun, M. Am. Soc. C. E. (by letter).—On May 5th, 1905, a suit was filed in the United States Circuit Court by the Porterville Water-Works Company *vs.* the City of Porterville. This suit has been instituted by the company to secure from the Court an equitable valuation of its water-works property, and to fix the rates to be received by the company upon this valuation. In this case there will be inquired into and brought out all questions of policy, design and construction, economy and management of the plant. The writer has been retained by the Porterville Water-Works Company; and the preparation, presentation and conduct of the case, as far as these matters are concerned, have been placed in his hands. In view of these facts the writer does not consider that professional

Mr. Harroun.

THE HERRON PERFORATOR,
FOR PERFORATING OIL OR WATER WELLS.



Scale of Inches

0 3 6 9 12

FIG. 5.

ethics will permit him to reply to the preceding discussions, and this he exceedingly regrets, as a number of points have been brought out which he desires to discuss. During the autumn of 1904 extensive changes were made, and the system was greatly enlarged, and many interesting and instructive data are available along many lines. It would afford the writer great pleasure to present these data, but, under the circumstances, he cannot consistently do so until the settlement of the suit now pending. Mr. Harroun.

Mr. Dunham asks regarding the method of perforating well casing in place. There is submitted a sketch, Fig. 5, showing the Herron Perforator, which is extensively used. On the right the perforator is shown in position, as it is lowered into the well. It will be observed that the collar supporting the punches is held down by a tie-wire passing through the tool near the bottom. This allows the punches to drop and miss contact with the casing. When the tool strikes the bottom of the well the trigger is forced against this tie-wire, cutting it and releasing the collar, which is then forced up by its spring against the punches, throwing them out in contact against the casing. The tool is now drawn up a few inches and then forced down slightly when the punches are forced out, as shown on the left. The operation is repeated throughout such sections of the casing as may be desired.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 995.

TECHNICAL METHODS OF RIVER IMPROVEMENT AS DEVELOPED ON THE LOWER MISSOURI RIVER, BY THE GENERAL GOVERNMENT, FROM 1876 TO 1903.*

BY S. WATERS FOX, M. AM. SOC. C. E.

WITH DISCUSSION BY

MESSRS. SAMUEL H. YONGE, H. M. CHITTENDEN, L. J. LE CONTE AND S. WATERS FOX.

INTRODUCTION.

When the first work on the Missouri River was undertaken by the General Government, very little was known of the physics of the river; and the only existing information as to works adapted to its regulation was of a negative character, previous efforts having been almost exclusively confined, and without successful results, to bank protection with solid, impermeable dikes.

The successful improvement of the river by regulation demanded, first, an intimate knowledge of the characteristic features of the river—of all the elements the sums of which are expressed in the varying conditions of the flow that obtain during a full cycle

* Presented at the meeting of March 1st, 1905.

of changes; and secondly, that a system of efficient, permanent works at moderate cost should be devised for the rectification and fixation of channel flow.

Necessarily, methods of treatment progressed step by step, with close observation and study of conditions of the flow, as found in their natural state and as affected by the works; and, meanwhile, an expectant but ever-impatient public found occasion, in the numerous failures which attended the earlier experimental works, to express disapproval and lack of confidence, not only in the works, but more especially in the extensive surveys, gaugings, etc., which, from an engineering point of view, were of such vital importance.

Before attempting a description of the more important works, it is thought best to give a brief description of the river.

Description of the River.

Formed by the confluence of three mountain streams—the Jefferson, Madison and Gallatin—the Missouri River, from its head (the mouth of the Gallatin), in latitude 45° 56′, longitude 110° 29′, to its mouth, in latitude 38° 49′, longitude 90° 7′, is 2 546.3 miles in length. It drains an area of 580 000 sq. miles, and its mean total annual discharge is estimated to be 20.15 cu. miles, or at a mean rate of 94 000 cu. ft. per sec.

The head of navigation is at Fort Benton, 2 284.8 miles above the mouth.

Above Carroll—2 113.2 miles above the mouth—the river is characterized as the "Rocky River," because of the nature of the bed in which it flows.

Below Carroll it gradually assumes character, first as a sand-bearer, and farther on, through tributary flow and change in bed formation, as a silt and sand-bearer, until, at Sioux City, Ia., 807.4 miles above the mouth, it is a pronounced type of alluvial river.

From Sioux City to the mouth, with an average slope of 0.86 ft. per mile, and an extreme gauge oscillation of nearly 19 ft. at Sioux City, and of nearly 35 ft. from Kansas City to the mouth, the river finds its way, in a tortuous, divided, ever-shifting course, down a valley composed of a heterogeneous mass of northern drift and al-

luvium. The bluffs on either side, defining the main valley, constitute the only natural barriers to the lateral migrations of the stream; and, but for some occasional formations of indurated clays and low-lying boulders, the underlying bed-rock is the only natural barrier against depth to which, in engorged sections, the river will scour.

From a maximum width of 17 miles between bluffs, for a short stretch below Sioux City, the width of the main valley varies considerably; on the whole, however, it becomes narrower below as well as above. At Yankton, S. Dak., the valley is 3 miles wide, and at Fort Randall about ¾ mile; at Omaha its width is 5 miles, and at Kansas City about 2 miles; between Kansas City and Glasgow the valley is wider, reaching a maximum of 8 or 9 miles in the vicinity of Carrollton, Mo.; from Glasgow to St. Charles the width is more uniform, and averages a little more than 2 miles.

The bluffs in the upper valley are covered with grass, except on the steeper slopes, which are bare; lower down, they are covered with undergrowths and forest trees. They form the sides of a great rock trough, the bottom of which, underlying the alluvium which at present partially fills it, is found at depths of from 70 to 100 ft. below the general level of the main valley.

The surface of the lower valley, in its uncultivated state, is covered with vegetation, from the thicket of young willows and cottonwood, on the low-lying bars and tow-heads, to large forest trees and mixed undergrowths which are found on the islands and main banks. Large numbers of trees are precipitated into the stream by caving banks every year. When their roots become embedded in the sand they form snags, and menace river craft. Sometimes they become the nuclei of wrack-heaps, and obstruct the flow to such an extent as to cause radical changes in the channel.

Numerous borings, subaqueous foundation works and other data, show that the bed of the river is composed of gravel and sand in all degrees of fineness—from very coarse to highly comminuted—and clays. Large quantities of vegetable matter are also found, from the tiniest rootlets to the largest forest trees.

While, in general effect, there is a sorting out of these materials, the heavier being found near bed-rock and the lighter near the surface, there is no regularity of formation. The heavier materials

are found at all elevations: in the shallow beds of high-water chutes, on the heads of bars, and even on the surface of high banks. The finer and lighter materials are found at great depths, as for instance, in old pools from which active flow has been suddenly diverted. Pockets, of greater or less area and thickness, at all depths and composed of any of the various materials, abound everywhere.

With such conditions of banks and bed, the difficulties of founding structures properly in the Missouri River must be apparent; and it will be understood why specifications should be drawn on broad lines and be of ready adaptability to local conditions.

Two floods occur every year with remarkable regularity—in April and June—and are known as the April and June floods, respectively. Both are destructive of property, and greatly disturb the channel.

The April flood is usually sharper than the June flood, and, when ponded back by ice, is very destructive, often producing the most astounding changes in channel alignment and location. The June flood, while usually higher, lasts longer, and, finding the channel in a measure prepared by the April flood, passes off with comparatively less damage to property and disturbance of channel.

For purpose of reference in establishing grades for river improvement works, bridges crossing the river, etc., the Missouri River Commission, in the fall of 1888, established two grade lines, from Sioux City to the mouth, respectively designated Standard High Water and Standard Low Water. The former was determined from the average of the highest known June floods to 1888; and the latter from the average of the lowest known stages to 1888 at which navigation was not prevented by ice.

The difference in elevation of the two planes at Sioux City is 10.42 ft., at Kansas City, 14.52 ft., at St. Charles, 16.13 ft.

Some idea of the effect of floods upon the regimen of flow may be formed from the fact that low-water discharge at a given stage—say standard low water—may vary as much as 100 per cent. It is approximately correct to state that standard low-water discharge at Sioux City may be anything from 15 000 to 30 000 cu. ft. per sec., and at St. Charles 20 000 to 40 000 cu. ft. per sec. To illustrate: at Sioux City the discharge in 1883 was nearly 70% larger than

that of 1879 at the same stage; while in 1895 it was more than 50% smaller than that of 1879 at the same stage; or, to give stage and discharge, there was at Sioux City in 1883 a discharge of 32 000 cu. ft. per sec. at a stage 1.5 ft. below standard low water, and in 1895, at a stage 1.5 ft. higher, a discharge of only 15 100 cu. ft. per sec., and, in a general way, the same conditions prevailed throughout the river, to its mouth.

Such a change of plane, involving a doubling of volume of flow at a given stage, can only be accounted for on the theory that, by extensive bed movement during flood stage, the efficiency of the channel was commensurately increased. The flood conditions preceding the two sets of measurements mentioned bear out this theory. Those of 1883 followed closely upon a flood which, in the lower river, was a phenomenal one, while the effects of the great flood of 1881, from Sioux City down, must have been still marked in the channel and on the planes of both the 1882 and 1883 low waters; on the other hand, those of 1895 were taken at the close of a long period of low water and when, for the three preceding years, flood volumes had been deficient in a marked degree.*

At standard high water the discharge at Sioux City approximates 200 000, and at St. Charles 300 000 cu. ft. per sec.

The greatest measured discharge of the river was 650 000 cu. ft. per sec. at St. Charles, taken on the crest of the flood of 1892.

The maximum discharge of the great flood of May and June, 1903, exceeded that of 1892, being estimated at 750 000 cu. ft. per sec.

By the almost constant erosion of banks and bed, quantities of the composing materials are carried forward by the stream, in suspension and in a semi-fluid state close to the bottom. Even during midwinter, when the river may be frozen over from source to mouth, and at extreme low stage, the water is never quite clear—never entirely free from sediment—while, in times of flood, immense quantities of sediment and drift-wood are borne along. It is estimated that every year the Missouri carries into the Mississippi enough sediment to cover a square mile 400 ft. deep. The character and quantity of the materials are such that if the current be checked a portion of it is dropped; and the rapidity and extent to which de-

* Report, Missouri River Commission, 1896, p. 3807.

FIG. 1.—METHOD USED IN BUILDING A CONTINUOUS ABATTIS OVER WATER. LOOKING INSHORE.

FIG. 2.—BANK-HEAD NEAR CHAMOIS, MO., LOOKING DOWN STREAM, AFTER STRUCTURE HAD BEEN EXPOSED TO THE FLOODS OF TWO SEASONS.

posits are thus formed would seem incredible to one unfamiliar with the river. On the other hand, an increase in velocity means greater sediment-carrying capacity and therefore greater tendency to scour. These features are strikingly illustrated, in the unimproved river, by the gorge sections which result from sharp impingement of flow against bluffs, and the inevitable wide flat reaches with middle-bar growths below. They are also well shown by the deposits formed above and below permeable dikes within the zone of slackened currents, and the trenches or pot-holes near the outer ends of these structures where the velocity has been increased.

As might be expected from such conditions, the course of the river is tortuous, and short reaches of sharply concentrated flow and great depth alternate with longer reaches of wide or divided flow and small depth. Innumerable bars, tow-heads and islands, snags and wrack-heaps obstruct its flow. While the general slope of the river, from Sioux City to the mouth, is uniform, it is broken at the pools and crossings, being less than the average in the pools and greater on the crossings. During a full cycle of change, from low water to flood stage and return to low water, the place of entry of principal flow into a pool, under normal conditions, will move from near the head of the pool toward its foot, and back again to near its head. It is during these changes that the banks in the bend—on the concave sides of pools—cave most rapidly, and that the channelways in the crossings through the intervening bars are most unstable and troublesome to navigation. On the rising stages of a flood there is increase in pool depth due to scour, and there is usually a compensating fill back on subsidence of the flood. In the crossings, however, the increase in depth does not keep pace with the increase in stage of the river; and a rapid decline in stage often leaves the crossing with no well-defined channel.

Improvement of the River.

The improvement of the Missouri River for purposes of navigation, as undertaken by the General Government, contemplated its regulation by restriction of the flow to widths, at high and low water, which would give desired depths at all stages, on fixed alignment. The channel was to be kept free from obstructions, such as snags and wrack-heaps.

The restriction of width was to be accomplished:

First, by Rectification, with Permeable Dikes, through the agency of which—in their function as deposit builders—flow was to be concentrated, old chutes or channels being closed, new banks built up to desired height and alignment, and the desired improved channelway developed.

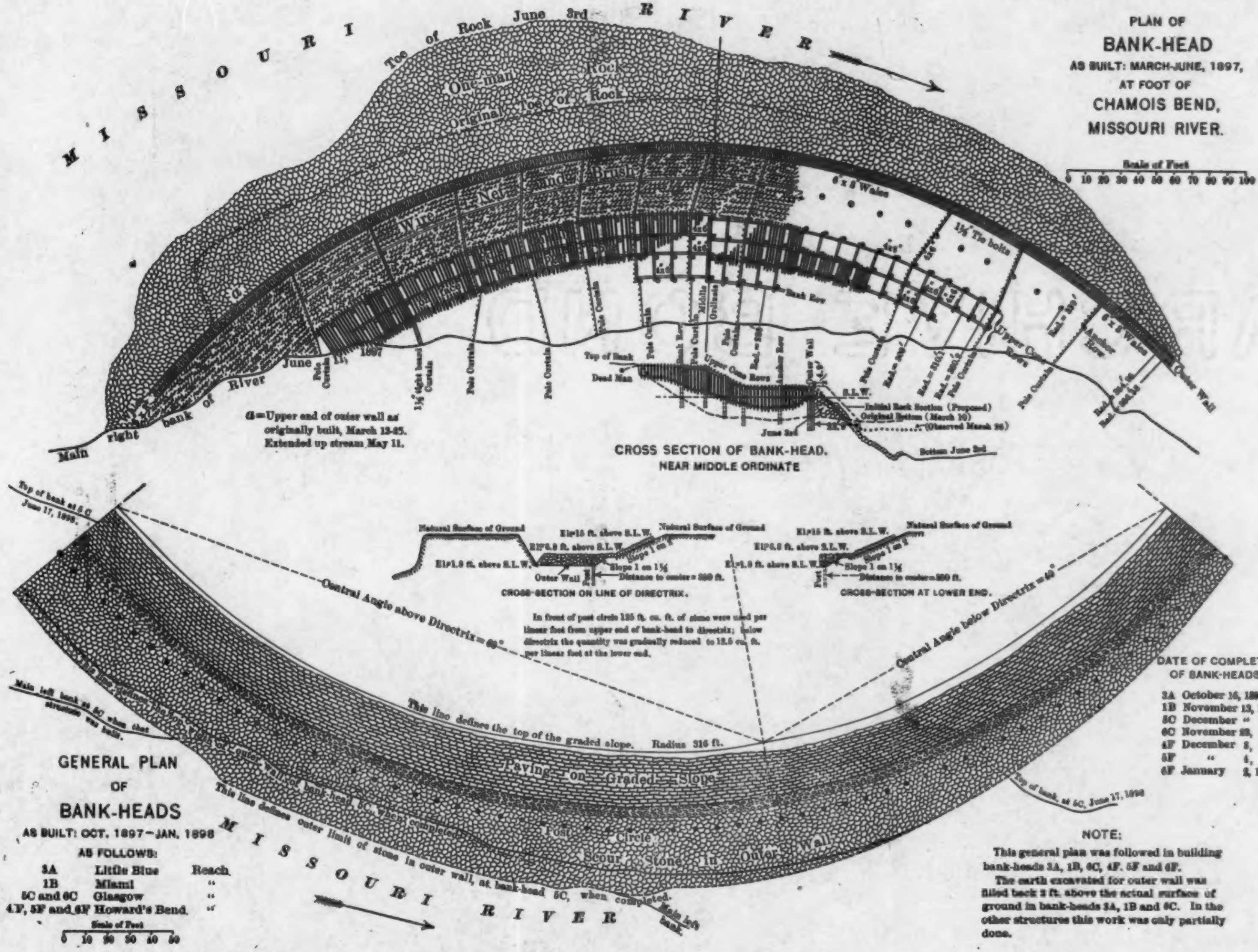
Second, by Fixation, with Bank Revetments, of Banks, whether natural or formed by rectification works, which conformed in position and alignment to the proposed banks of the improved river.

The history of the methods and devices used in the development of Missouri River improvement works embraces the period of twenty-seven years following 1876. It is full of interest to the engineer, contains much of original and scientific research, and has contributed in no small degree to the sum of present knowledge concerning the flow and proper treatment of detrital rivers. Its narration here cannot be attempted, but a brief outline is given, with a view of indicating the range covered by experimentation.

Permeable Dikes.—The earliest permeable dikes on the Missouri River were composed of one or more rows of "Brownlow Weeds" in variously modified forms. In its simplest form the weed was composed of single pieces, or bundles, of brush trailing from a rope or cable, one end of the rope being anchored at the bottom and the other end supported at the water surface. The butt ends of the brush were attached to the rope at short intervals from bottom to surface, the other ends being free to trail down stream as impelled by the current. The length of the rope was from one and one-half to four times the depth, according to the exposure and the existing stage of the river. In a more elaborate construction, the weed resembled a series of umbrellas, formed by brush radiating densely from a central fascine, the latter being anchored at one end at the bottom, and supported at the other end on the surface of the water by a buoy.

The results obtained with weeds were instructive, but, on the whole, unsatisfactory, and, after a thorough trial, they were abandoned. Deposits covering large areas were quickly formed by them, but at elevations considerably lower than the highest stage to which the weeds were exposed, and most often they were not distributed uniformly, being broken by small waterways. Constant rotation

PLATE XXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 995.
FOX ON
MISSOURI RIVER IMPROVEMENTS.



or swirling often destroyed or detached the weeds; loss of, or leaking, buoys, a dragging down by drift-wood and finer vegetable growths, as well as by deposits, made others inoperative.

Buoyed curtains, or sectional gratings of brush or poles, anchored and buoyed in positions imbricating down stream, were also tried without success.

Later, extensive experiments were made with wire netting, of various forms and areas of mesh, and much ingenuity was displayed in its manufacture on the work by specially designed machines.

Dikes formed of a continuous piece of netting, anchored and buoyed in position, gave results which were little if any better than those obtained with weeds. Fixed supports were then substituted for buoys, with marked improvement, but developed the weak points of the netting. When held rigidly in the current, the netting was likely to be punctured and torn by large pieces of drift-wood, or, being strong enough to resist rupture, would bag down stream under the strain, thus inviting the accumulation of drift-wood which sooner or later destroyed it. But, what is of more importance, the trial of wire netting with fixed supports developed the necessity for provision against scour, many of the failures being due to that cause.

One of the earliest forms of fixed supports was a tripod, composed of heavy poles—the spread at the base being proportioned to the depth—and, for increased stability, weighted with stone confined in pieces of wire netting. The tripods were dropped into position—from 20 to 50 ft. apart on the line of the dike—from a barge by a derrick. Later, piles were used: single piles in A-bents and vertical, and piles in clusters. Then followed in rapid succession: uniform and shorter spacing of vertical piles in single or multiple rows; the introduction of wales and braces for the transmission of strains; the use of foot-mattresses for protection against scour; and the substitution of gratings or curtains of poles for wire netting.

Thus was delevoped the form of permeable pile dike used so extensively in the systematic improvement of the river, and which is described in detail farther on.

Bank Revetments.—The continuous, woven revetment, in its

present form, as adapted for the protection of banks against erosion, is the outgrowth of years of experimentation:

First, with sectional mattresses of various kinds extending from the water's edge at the existing stage of the river 60 to 100 ft. into the stream, the upper bank being graded to a steep slope and covered with a thatching of brush, the latter held in place by wire or wire netting pinned to the slope.

The mattresses were composed of a grillage of brush held in place by top and bottom frames of poles or planks, the frames being fastened to each other through the mattress with pins and wedges, or wires, or both. In some cases an intermediate filling of grasses was put in. The first mattresses were made on fixed ways built in the bank at the head of the proposed revetment. As each section of mattress was completed, it was launched from the ways, allowed to make a quarter turn in the current, and then floated down to its position in the revetment, the first one going to the lower end of the work, the next one overlapping its up-stream edge, and so on, until the desired length of bank had been thus covered. Frequent loss of mattress, in transit from the ways to position in place, led to placing the ways on boats, from which the mattresses were launched directly into place.

Numerous failures of revetments made with sectional mattresses were found to be due, primarily, to their slipping or displacement from scour or other causes, thus breaking the continuity of the work and exposing all below it to loss by flanking. This consideration led to the adoption of continuous revetment. In one of the earliest forms, a continuous mattress, extending from the water's edge 112 ft. into the water, was made of a grillage of brush extensively sewed with wire and reinforced longitudinally with continuous brush fascines; the latter were sewed on top of the mattress, 5 ft. apart, in lines parallel to the outer edge. The first or bottom course of the mattress was made of willow brush, as compact as possible, laid across the ways, and normal to the bank line; the next or middle course was of scraggy, dogwood brush laid at right angles to the brush in the first course; and the third or top course was of willow brush laid across the brush in the middle course, only enough being used to bind and hold the latter in place. Thus, in effect, a continuous brush carpet was made, the warp, so to speak, being close

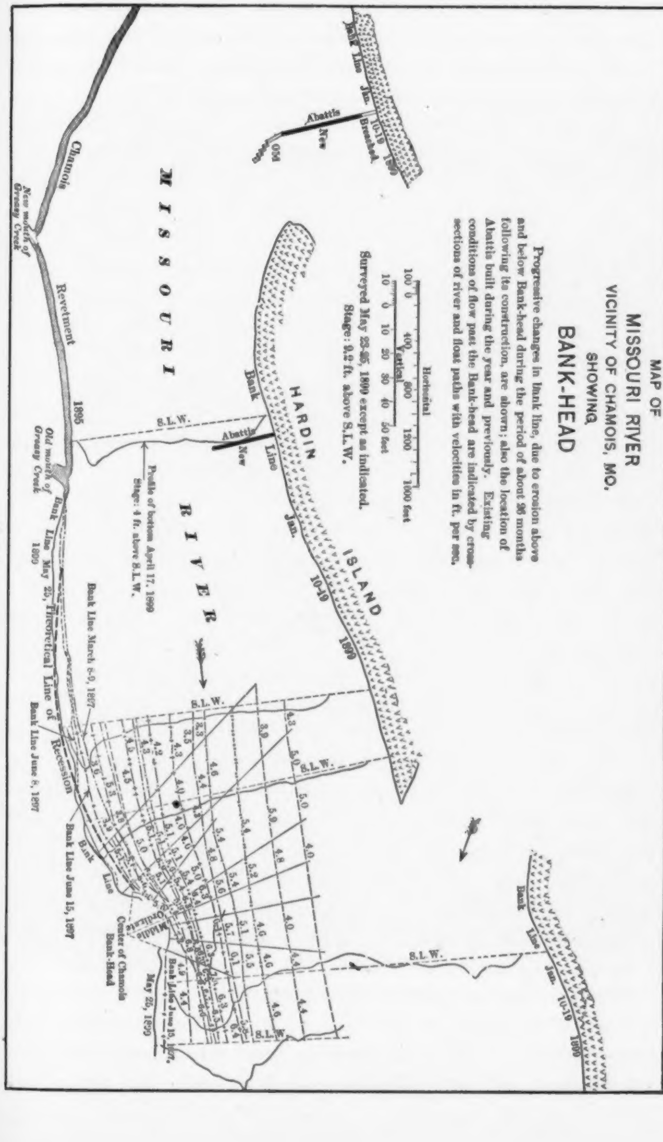

MAP OF
MISSOURI RIVER
VICINITY OF CHAMOIS, MO.
SHOWING
BANK-HEAD
Progressive changes in bank line, due to erosion above and below Bank-head during the period of about 26 months following its construction, are shown; also the location of Abattis built during the year and previously. Existing conditions of flow past the Bank-head are indicated by cross-sections of river and float paths with velocities in ft. per sec.
Horizontal
Vertical
Surveyed May 23-25, 1899 except as indicated.
Stage: 9.2 ft. above S.L.W.
MISSOURI
RIVER
HARDIN
ISLAND
Chamois
New mouth of Greasy Creek
Old mouth of Greasy Creek
Revetment
Abattis
New
Old
S.L.W.
Profile of bottom April 17, 1899
Stage: 4 ft. above S.L.W.
Bank Line March 6-9, 1897
Bank Line June 6, 1897
Bank Line June 15, 1897
Bank Line May 25, 1899
Theoretical Line of Recession
Center of Chamois Bank-Head
Bank Line Jan. 10-19 1899

FIG. 1.

and compact, the woof, open and cellular. The mattress was built from the upper end down stream, on a boat provided with launching ways, from which the mattress was launched as made. Stone, about $\frac{1}{3}$ cu. yd. per lin. ft., was used to sink the mattress.

The upper bank was graded to a slope of 1 on $2\frac{1}{2}$ and covered with a layer of brush 8 to 10 in. thick; a covering of earth about 8 in. thick was put on the latter, with a view of encouraging the growth of willows, and to protect it against loss by fire and abrasion by ice.

The particular piece of work here described was built in the fall of 1880, and, in the following spring, was subjected, without damage of any kind, to as severe an attack as any revetment could be, by the great flood of 1881. It is still intact, though for many years it has been masked by a shore bar.

Actuated by the thought that possibly such work was much heavier than necessary, some brief and convincing experiments were made with light, open works: A continuous wire netting, having rectangular meshes $2\frac{1}{2}$ by 5 ft., in each of which was fastened a piece of scraggy, dogwood brush, was made in one piece, extending from the top of the bank, on a graded slope of 1 on $1\frac{1}{2}$, about 80 ft. into the stream. The bank edge of the netting was fastened to stakes driven 5 ft. apart on top of the bank, and only $1\frac{3}{4}$ cu. yd. of stone were used in sinking 100 lin. ft. of it. This and similar constructions were quickly destroyed by the river.

Then the continuous woven mattress, first used on the Missouri River at Vermilion, S. Dak., came into general use. In early practice, the woven mattress, without reinforcement of any kind, extended from the top of the bank, on a graded slope, various distances into the stream. At one time the width of the mattress was such that, when in contact with the bottom, its outer edge would lie on the return slope, beyond the thalweg. Later, the mattress was strengthened longitudinally and transversely, first with large steel wires and subsequently with strands of smaller wires; and the transverse members were used to anchor the mattress to the bank.

Then the woven mattress was omitted from the upper bank, its inshore selvage lying at the foot of the graded slope just above water, and being held or anchored by short piles, spaced from 8 to $16\frac{2}{3}$ ft. apart, with and without connecting wales. The upper bank

BANK-HEAD NEAR CHAMOIS, MO., NEARING COMPLETION. LOOKING DOWN STREAM.

was protected with rip-rap, thrown on so as to cover it well. The next and final steps completed the development of what is now known as the standard revetment, and which is described in detail farther on.

In 1897 two new devices were introduced, both of which were designed by Colonel Amos Stickney, Corps of Engineers, at that time President of the Missouri River Commission.

One of these was a silt-catching device, called an "abattis" because of its similarity in form to the device of that name used in defensive military operations. It was first used as an auxiliary means of closing chutes, and proved so effectual that, later, it was also used for the advancement of shore lines in situations not exposed to the full forces of the river. Fig. 1, Plate XXI, shows the design and application of abattis.

The other device, called a "bank-head," was for use as a means of bank protection. The theory of the bank-head system was, that by the fixation of the bank at points some distance apart, the bank between those points would become stable after receding to a certain line, within reasonable, practicable limits; and that the resulting conditions of the flow in front of the structures and in the approaches would be free from objectionable features. The distance between fixed points was to be determined by experiment, but it was held that banks could be protected by bank-heads at a cost of one-third to one-fifth of that of revetment work.

The first bank-head was built in March, 1897, at the foot of Chamois Bend, 3 850 ft. below the lower end of a continuous revetment, the latter being 2¾ miles in length.

The arrangement of the structure is shown in plan and cross-section on Plate XXII. The progressive changes in bank line due to erosion above and below the structure during the period of 26 months following its construction, and the resulting conditions of flow past it are shown in Fig. 1.

Plate XXIII shows the bank-head nearing completion, and Fig. 2, Plate XXI, is a photograph taken after it had been exposed to the floods of two seasons. In form, the structure consists of segments of the frusta of two cones having a common axis, the lower one resting against the bank and on the bottom of the river, its top cut by a plane at an elevation of 6 ft. above standard low water, and

developing an arc of a circle 724.4 ft. in diameter and having a middle ordinate from the bank line of about 90 ft.; the upper frustum resting against the bank and on the lower one, developing an arc of a circle on the plane of contact therewith 640.4 ft. in diameter and from a common center. The top of the upper frustum is defined by the height of the river bank. The conical surface of the lower frustum is defined by the stone of the outer wall; that of the upper one by the pole screening.

Both surfaces, of course, were subject to change: the former from movement of the stone due to scour, which might finally leave the vertical wall of piles; the latter from accretions formed during stages of river in excess of 6 ft. above standard low water. In all, 608 piles were driven, as follows: 446 in the outer wall, 29 in the anchor row, 9 in the bank row, 109 in the upper-cone frustum, and 15 for temporary outside anchors. The piles in the outer row were driven to an average penetration of 22¾ ft., and all others to 20½ ft. It was intended that the piles in the outer wall should be driven 22 in. between centers in the rows, and that the rows should be 19 in. from center to center; but, because of the packing of the ground, due to jar as well as displacement, it was found impossible to drive them to the desired depth so close together. The distance apart in the rows, therefore, was increased to 3 ft. from center to center of piles, and brush was introduced between the piles, with a view of producing a practically tight wall. The specifications called for 3 400 cu. yd. of one-man stone in front of the wall. Of this quantity, 2 467 cu. yd. were to be placed above the middle ordinate, in a practically uniform section of 250 sq. ft., and the remainder below, in diminishing section. In placing this, an effort was made, first to get on the bottom, as quickly as possible after the outer row of piles had been driven, a layer of stone 2.5 ft. thick and 22.9 ft. wide above the middle ordinate and diminishing from there below; thereafter the requisite quantity of stone per linear foot was simply thrown against the piles in the outer circle and allowed to find its slope. The space inside of the outer circle and the upper-cone frustum was to be filled by deposit from flood. For that purpose 1 497 lin. ft., or about 31 600 sq. ft., of curtains were built. One of these, on a line 30° up stream from the middle ordinate, was a tight curtain, made of 1-in. boards. The horizontal

FIG. 1.—BANK-HEAD ABOVE ROCHEPORT, MO., AS REMODELED. LOOKING DOWN STREAM.

FIG. 2.—BANK-HEAD NEAR MIAMI, MO., AFTER REMODELING AND EXTENDING. LOOKING DOWN STREAM.

curtain, extending from the outer wall to the base of the upper-cone frustum, was made of wire netting, the meshes of which were practically filled with brush. The first cost of the bank-head was $11 265.13.

It was expected that the bank-head would hold the bank permanently at that point, and that, after a certain amount of recession, the stretch of free bank above it would become stable. It was expected that a scour in front of the bank-head would occur, and, based upon some experiments made with a model, the quantity of stone put in the outer wall was thought to be sufficient to follow up a scour of 50 ft., and that the stone would cover the resulting slope of the river bed so as to protect it against erosion. The form given the structure was thought to be such as would prevent, at all stages, the formation of violent eddies. During the period of fourteen months following the construction of the bank-head the bank above it continued to cave, the area of greatest erosion lying within a distance of 1 100 ft. above the structure. For about 1 000 ft. above the bank-head the shore line had receded back of the theoretical line of recession, forming a deep bay which terminated in a shoulder at the bank-head. At the latter point, measuring on the line of the outer wall of the bank-head, the bank had receded about 240 ft. This portion of the bank (for a distance of 1 100 ft. above the bank-head) then became comparatively stable until the fall of 1902, though, meanwhile, there was some bank caving all along from the bank-head to the foot of the revetment. During the two years following its construction, the bank-head was closely observed, and such repairs, reinforcement and extension as seemed necessary were made. The last extension of the outer wall, 44 ft. in length, was made in August, 1897, increasing the central angle of the structure to 103° 26′; but, by June, 1898, the bank had receded until a further extension of 110 ft. would have been necessary to connect the outer wall with the bank. This would have made the central angle of the bank-head 121° 16′. In March, 1899, two subaqueous spurs or groynes were added to the bank-head, extending from the outer wall into the stream about 150 ft. on radial lines 30° and 41° 27′, respectively, above the middle ordinate. The object of the spurs was to fill up the trench around the bank-head and prevent its formation again. For a short time the conditions were im-

proved by the spurs, but they soon settled, and their effect was lost. In the fall of 1902 there was some caving of the bank above the bank-head, and by the following spring the shoulder immediately above it had deepened to such an extent that the structure was seriously threatened. Two spurs, built against the bank at the head of the structure, held it until the great flood of June, 1903, when the bank-head was flanked and destroyed.

During the fiscal year ending June 30th, 1898, nine other bank-heads were built in the Missouri River, as follows:

Three in Howard's Bend, above St. Charles, Mo., one above Rocheport, Mo., three above Glasgow, Mo., one near Miami, Mo., and one in Little Blue Reach, above Missouri City, Mo. Another bank-head, making eleven in all, was partially built at the foot of Little Missouri Bend, above Glasgow. It was never completed, and, though not in the line of flow, has been practically destroyed. The bank-head above Rocheport was destroyed in June, 1902.

With the exception of the Chamois bank-head, already described, the one in Little Missouri Bend and the structure above Rocheport, all these bank-heads were first built under a general plan, as shown on Plate XXII. It will be noticed that, while the form was similar to that of the Chamois bank-head, the use of piles was done away with, stone only being used, and the structure was set back so that when completed it was almost entirely masked by the bank. It was thought that, until it was uncovered by erosion, its effect upon the flow would be gradual, permitting the river to accommodate itself to the restraint without producing any violent change of regimen. The floor of the rock wall was excavated to the level of standard low water, as nearly as practicable; the inner line of the floor was an arc of a circle of 350 ft. radius, located so as to be tangent to the standard low-water contour of the bank. This line was defined on the ground, for guidance during construction and for subsequent reference, by 6 by 8-in. posts, 10 ft. long, driven 10 ft. apart on the circle, and to a depth of about 5 ft., or until their tops were at or near 5 ft. above standard low water. Above the directrix, the floor was excavated to a width of 27½ ft., where it fell entirely within the bank; in front of the post circle, on the floor where it fell within the bank and in the water or the foreshore outside of the bank, 125 cu. ft. of rock per linear foot were placed;

FIG. 1.—DIKE 1 B, PRACTICALLY COMPLETED.

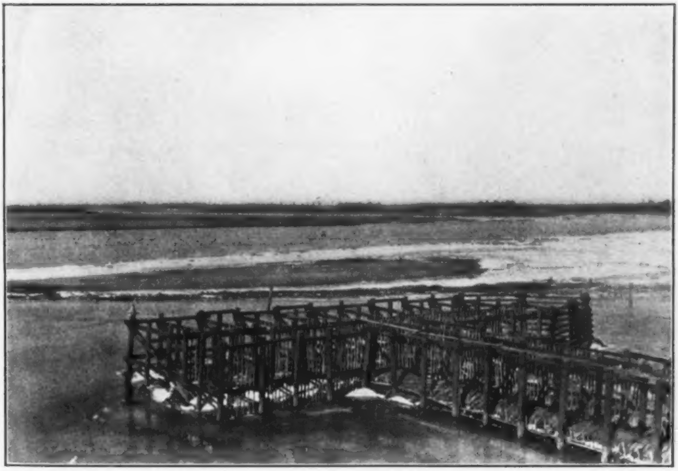


FIG. 2.—T-DIKE, IN WILHOITE BEND.

below the directrix, the quantity of rock was gradually diminished to 12.5 cu. ft. per linear foot at the lower end, the width of the excavation varying to give the specified cross-sectional area within the uniform depth of 5 ft. for the rock wall. Inside the post circle the bank was graded to a slope of 1 on 1½ to an elevation of 4 ft. above standard low water, then a 4-ft. berm was made, from which the bank rose to the surface of the ground on a graded slope of 1 on 2; the space back of the post was filled with stone, and the berm and slope were paved to a thickness of 1 ft., the latter up to a height of from 15 to 20 ft. above standard low water. After the stone was in place, the excavation was generally filled back to an elevation about 2 ft. higher than the natural surface of the ground, and the remaining earth excavated was wasted in front of the bank-head.

A somewhat modified form of construction was adopted in building the bank-head above Rocheport, *viz.:* The excavation was made entirely within the bank; its floor, or the bottom of the wall, was made 15 ft. wide and at an elevation of 10 ft. above standard low water; the outside line of the floor was an arc of a circle of 315 ft. radius, located tangentially to the 10-ft. contour of the shore; a section of the rock wall was a parallelogram, 150 sq. ft. in area throughout, 15 ft. wide on top and bottom, 10 ft. high, with side slopes of 1 on 1. Fig. 1, Plate XXIV, is a photograph of this bank-head as subsequently remodelled.

Fig. 2, Plate XXIV, is a photograph of Bank-head 1 B, near Miami, Mo., as subsequently remodelled and extended.

It developed early in the history of these structures that, in their original form, they would not withstand sharp attack by the river. All but two of them, in the upper part of Howard's Bend, were remodelled and, subsequently, extended, reinforced and repaired; and, in the majority of cases, auxiliary structures, in the way of pile dikes and abattis, were built to protect the bank-heads and to ameliorate undesirable conditions of flow past them.

By these means, involving an expenditure of a sum of money largely in excess of original estimates, and the loss, meanwhile, of considerable land, the three bank-heads in Wilhoite Bend, above Glasgow, were maintained, and conditions of flow developed that, for a time, were highly satisfactory. Fig. 2 is a map of the Missouri River, showing the system of bank-heads, and Fig. 3 shows the results of treatment by this method.

There was reason for belief that, after some further experimentation, on lines suggested by the experience gained, a satisfactory system of bank-head protection could be devised; but, at this juncture, appropriations failed, and the ultimate success of bank-heads, as a practical, economical means of bank protection on the Missouri River, yet remains to be demonstrated.

FIG. 2.

Description of More Important Dikes and Revetment.

Systematic Improvement.—The most important work on the Missouri River is that done since 1891 by the Missouri River Commission in the systematic improvement of 45 miles of the upper portion of First Reach, extending from 4 miles above Jefferson City to the mouth of Gasconade River, the latter being 110 miles above the mouth of the Missouri.

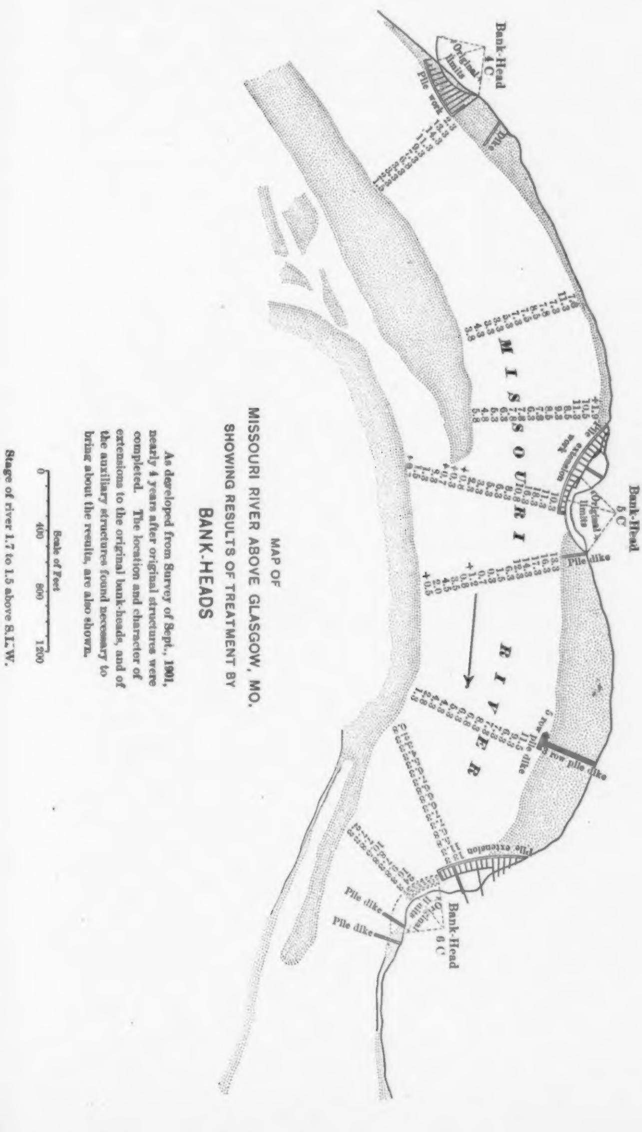

MAP OF
MISSOURI RIVER ABOVE GLASGOW, MO.
SHOWING RESULTS OF TREATMENT BY
BANK-HEADS
As developed from Survey of Sept., 1901, nearly 4 years after original structures were completed. The location and character of extensions to the auxiliary structures found necessary to bring about the results, are also shown.
Scale of Feet
0
400
800
1200
Stage of river 1.7 to 1.5 above S.L.W.
All soundings reduced to S.L.W.
Bank-Head 4 C
Original limits
Pile work
Dike
MISSOURI RIVER
Pile extension
Bank-Head 5 C
Original limits
Pile dike
5 row pile dike
3 row pile dike
Pile extension
Bank-Head 6 C
Pile dike
Pile dike

FIG. 8.

This was one of the worst reaches on the river, presenting, not only a category of characteristic conditions, but especially difficult conditions at the mouth of Osage River.

The proposed widths of the improved river were: Above Portland, 1 000 ft. between standard low-water contours, and 1 140 ft. between standard high-water contours; below Portland, 1 100 ft. and 1 240 ft., respectively.

The total cost of the improvement of the 45 miles of river was $2 500 000, or a little less than $56 000 per mile.

The results obtained were highly satisfactory. A continuous channel, conforming closely to project lines, and with a minimum low-water depth increased from 2½ to 6 ft., was made; and new banks were aligned and well advanced in the process of building up to desired heights. Incidentally, large areas of land were reclaimed, to say nothing of land protected.

Although the works and results have suffered by neglect, they amply justify the statement that the improvement of the river is entirely feasible, and at a cost per mile much less than that which has been improved. "The work at the mouth of Osage River was exceedingly difficult and costly; nothing like it is to be expected at any other point on the river." Furthermore, "the work was largely experimental, and the experience gained will unquestionably render possible a very considerable reduction, both in the extent and cost of future work."

Permeable Pile Dikes.—These structures consist essentially of one or more rows of piles, as indicated by the exposure to which the dike will be subjected, the piles being driven, or sunk, to such penetrations as will, with a given system of bracing for the transmission of strains, develop their full strength; provided with a foot-mattress for protection against scour; and screened or curtained to check flow through it and thereby cause deposit.

When applied to the advancement of a bank out to the proposed line of rectification, these structures are built from the existing bank out, on a line bearing slightly down stream from a normal to the flow. The practice, as to height of dikes, differed on the two divisions, and varied from time to time; but, in the main, they were run out level with the top of the bank, or 2 ft. above standard high water, to near the standard high-water contour of the proposed recti-

FIG. 1.—PILE-DIKE, OPPOSITE CHAMOIS, MO., WHILE IN FLOOD, SOON AFTER COMPLETION.

FIG. 2.—PILE-DIKE, OPPOSITE CHAMOIS, MO., SHOWING ACCRETIONS FORMED BY THE DIKE WITHIN A FEW DAYS AFTER COMPLETION.

fied shore, and from there they sloped down to as near 2 ft. above standard low water as the stage of river at the time permitted.

When located on the bar side of the river, or the convex side of bends, the exposure is generally, or at least frequently, such that the dike is made of increasing strength from the bank out, *i. e.*, commencing at the bank with 1-row work and changing successively to 2-row, 3-row and 4-row work as the exposure increases with distance from the bank. When located on the concave side of bends, the dikes, on account of the exposure, are seldom of lighter construction (fewer rows of piling) at their bank ends, than at their stream or outer ends; and in some instances the heaviest construction is near the bank. This was found necessary wherever the thalweg was close to the bank, and pile penetrations were limited by excessive depth of water, impenetrable bottom, or other cause. The lower dikes of a group of dikes were made of lighter construction than the one farthest up stream, excepting that portion of each which projected beyond the influence of the dike next above it. The outer ends of all dikes, for a length of from 50 to 200 ft., were reinforced: usually by additional piles driven alongside the upper pile of each bent; by additional top diagonal braces; and by increasing the width of the foot-mattress from 5 to 10 ft. on the up-stream side. In order to strengthen still more the outer ends of dikes, to ameliorate the eddy action there and to cause the deposit to form as far out as possible, all dikes were provided with a head, of one of the following forms: A wing or trail of 2-row pilework, similar in construction to the main structure and extending down stream from 60 to 100 ft. or more; T-shaped heads of like construction extending above as well as below the dike; heads formed by a cluster of three piles 20 ft. above and another cluster 20 ft. below the stream-end bent space, the clusters being connected to the outer two bents of the dike by wales.

Fig. 1, Plate XXV, shows one of a group of dikes built from the concave side of the river where the penetration of the piles for some distance from the bank was limited by bed-rock. It is provided at its stream end with a 2-row T-head. Some views of other dikes are given on Plates XXVI and XXVII.

When located within a chute, for the purpose of closing it, the dike, of course, extends across the chute from bank to bank at bank

height; and when, as is generally the case, the structure is exposed to active high-water flow, the upper dike, if more than one be used, is very substantially built. Especial attention is given to make the shore ends of chute dikes secure against flanking by erosion which may develop in the event of the release of head on the structure. In especially difficult situations, or where the head upon the structure would otherwise be very large, the spacing between the poles of the curtain is rapidly increased from both banks toward mid-width, or, the curtain is omitted for several bents near the middle of the structure.

The specifications which governed the dikes constructed on the two divisions of First Reach differed in details, and were changed from time to time. The following specifications, drawn up on June 23d, 1896, embody details, some of which were peculiar to the practice on each division and some common to both. As a whole, they represent what was considered at that time the best practice:

"SPECIFICATIONS."

"The most natural sequence in which the different branches of dike construction are carried on, is as follows: Pile-driving, construction of foot-mattress and sinking of the same, bracing, screening and lashing. The specifications for each of these operations are given below in the order named:

"*Pile-driving.*—The piles shall be driven in rows 10 ft. between centers, the rows being 10 ft. between centers. The number of rows shall be from one to five according to the anticipated exposure, or limitation of penetration. All piles shall be driven to a penetration of not less than 25 ft., unless limited by bed-rock. In cases where bed-rock limits penetrations to less than 20 ft., the desired stability is to be secured by increasing the number of rows, or, when limited to less than 15 ft., by filling in with stone. In special cases where excessive scour or great stress is anticipated, as at the stream ends of dikes on a concave shore, or the upper dike of any group, penetrations up to 35 ft. may be necessary.

"For 10 bents inshore from the head, the upper row of piling should be reinforced at each bent by an additional pile driven close and lashed to it.

"Piling shall be of such lengths that when driven to their specified penetrations their tops shall be not more than 1 ft. below or above grade.

FIG. 1.—LITTLE TAVERN DIKES, NEARING COMPLETION.

FIG. 2.—ACCRETIONS DUE TO LITTLE TAVERN DIKES, AS THEY APPEARED TWO YEARS AFTER COMPLETION OF THE DIKES.

"*Foot-mattress.*—The foot-mattress shall be of standard type, compactly woven, and not less than 12 in. thick, the width of mattress and its reinforcement by galvanized steel strand being as specified below.

"The width shall be 30 ft. for 1-row dike, 40 ft. for 2-row, 55 ft. for 3-row, and 65 ft. for 4-row, and shall be increased 10 ft. for each row in excess of 4-row. The upper selvage edge must extend 20 ft. above the upper row of 1- and 2-row work, and 25 ft. above for 3-row work and upward, the above widths being increased 10 ft. on the upstream side of the dikes for a distance of 100 ft. from their outer ends.

"The foot-mattress must extend from the bank at the shore end, at an elevation there of 6 ft. above S. L. W., to 40 ft. beyond the outer row of piles of the dike head at the stream end, and along the head 20 ft. beyond each of its extremities. The width of mat at the head shall be 70 ft. The mattress shall be reinforced by $\frac{3}{8}$-in. strand longitudinally and transversely, as follows: One-row work shall have a single strand in each selvage, and a single longitudinal 8 ft. above the piles; 2-, 3- and 4-row work shall have a single strand in each selvage and a single longitudinal 8 ft. above the upper row of piles; all mattress shall have transversals of double $\frac{3}{8}$-in. strand across the mat from selvage to selvage, midway of alternate bents, the transversals being pin-twisted or toggled at the intersections of longitudinals and on the lines of piling, and the longitudinals being pin-twisted or toggled at intersections with transversals and at intermediate points between transversals. The same spacing of strand and other specifications shall apply to the mattress in and beyond the dike head.

"The mattress shall be sunk to close contact with the bottom throughout, a sufficient quantity of ballast being used to insure this. In no case shall a less quantity than $\frac{1}{2}$ cu. yd. per 100 sq. ft. of mattress be used.

"In addition to the stone above specified, 50 cu. yd. shall be distributed on the outer 100 ft. of the main dike and 25 cu. yd. on each 100 ft. of the dike head, the stone ballast being in fragments weighing not less than 20 nor more than 60 lb. and no stone being less than 3 in. thick.

"For temporary anchorage during construction, single piles shall be placed not less than 60 ft. above the upper row of dike piles, the distance between the piles being governed by the depth of water and velocity of current.

"*Bracing.*—The bracing system shall consist of wales placed longitudinally, and braces placed transversely to the dike, the latter being divided into three classes, *viz.:* 'direct transversals,' 'diagonal transversals' and 'drop transversals.' All bracing shall be long-leaf

yellow pine. The wales are to be bolted on the up-stream side of each row of piles, and are to be uniformly 6 by 8 in., and 22 ft. long.

"Direct transversals, consisting of two pieces, 3 by 8 in. by 22 ft., for the upper three rows, and 3 by 8 in. by 12 ft. or 22 ft., for additional rows, as may be necessary, shall be bolted to the sides of each pile.

"The direct transversals shall be gained 1 in. at each end to take the thrust from the upper and transmit it to the lower wale.

"In the construction of dikes located on cross-sections of river, the high-water widths of which are 3 000 ft. or less, the direct transversals will be placed under the wales; for dikes beyond the 3 000-ft. limit they will be placed on top of the wales.

"Diagonal transversals, 5 by 8 in. by 16 ft., shall be drift-bolted to the tops of the piling. These braces are to be laid on the upper three rows, forming a single intersection for the full length of the upper dike of all groups, or, for isolated dikes. They are also to be used for the outer 150 ft. of all dikes, and for that portion of a dike on a concave shore which projects beyond the dike next above. These diagonal transversals are to be also used for bracing the dike head.

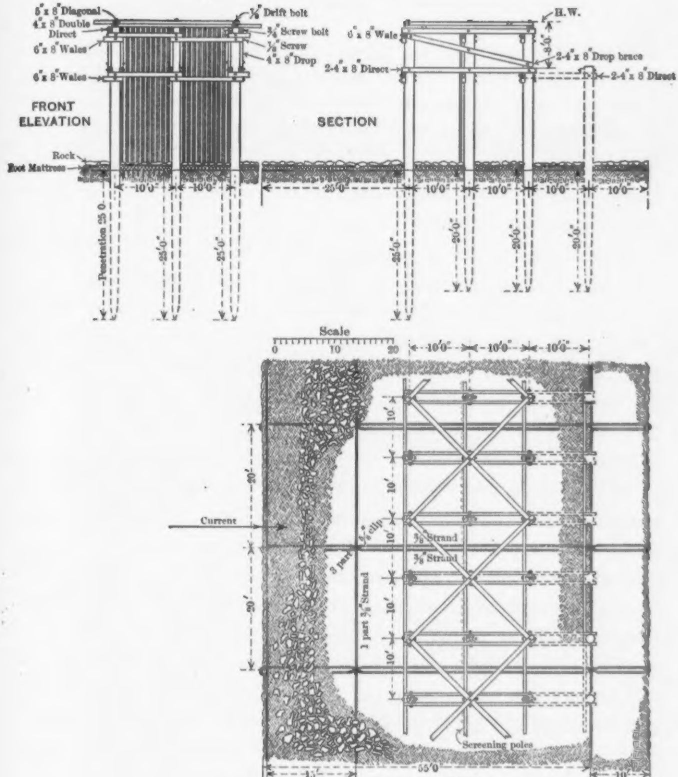
"For dikes outside the 3 000-ft. limit a double system of bracing shall be used, the upper of which is to be in all respects similar to that described above for dikes within the 3 000-ft. limit, with the exception already noted, as to the relative positions of the direct transversals and wales, and the further exception that the diagonal transversals are to be omitted. The members of the lower system are to be the same as those of the upper system, and placed 8 ft. below them. A drop-transversal, consisting of two pieces, 3 by 6 in. by 24 ft., extending from the top wale of the upper row to the lower transversal of the third row, is to be introduced at each bent and bolted to each pile.

"All wales and braces, except diagonal transversals, are to be fastened to the piles with $\frac{7}{8}$-in. screw-bolts. The diagonal transversals are to be fastened with $\frac{3}{4}$-in. drift-bolts.

"*Lashing.*—The top system of bracing shall be lashed together at the piles with $\frac{3}{8}$-in. galvanized-steel strand throughout the entire length of all upper dikes, and on their heads, and on all other dikes and their heads, wherever diagonal transversals are used.

"*Screen-Work.*—Screens shall be formed of poles placed on the next to the lower row of dike piling and middle row of dike-head piling. The poles are to have butt diameters not less than $2\frac{3}{4}$ in. nor more than 5 in. The spaces in the clear between adjacent poles must vary from 2 to 6 in. The poles are to be sharpened, driven well through the mat, and fastened to the up-stream side of the wale with wire nails.

"When additional supports for the screen poles are required, on



TYPICAL 3-AND 4-ROW DIKE AS USED 1891 TO 1897.

The latter being in all respects like the former.
With additional work shown in dotted lines.
In low dikes, or those not greatly exposed,
the lower system of bracing is omitted.

FIG. 4.

account of excessive depth or current velocities, such supports are to be given either by ⅜-in. wire strand or poles fastened to the piling.

"In cases where circumstances prevent the procuring or use of poles for screen work, wire netting, placed in front of the upper line of piling, and with area of mesh of about 24 sq. in., shall be used.

"After the outline of the accretions becomes defined, or within a period of three years after the completion of the dike, that portion of the dike beyond the accretions shall be reinforced by filling in with mattress and stone, to form a submerged spur extending on an approximately uniform slope about 40 ft. beyond the dike head."

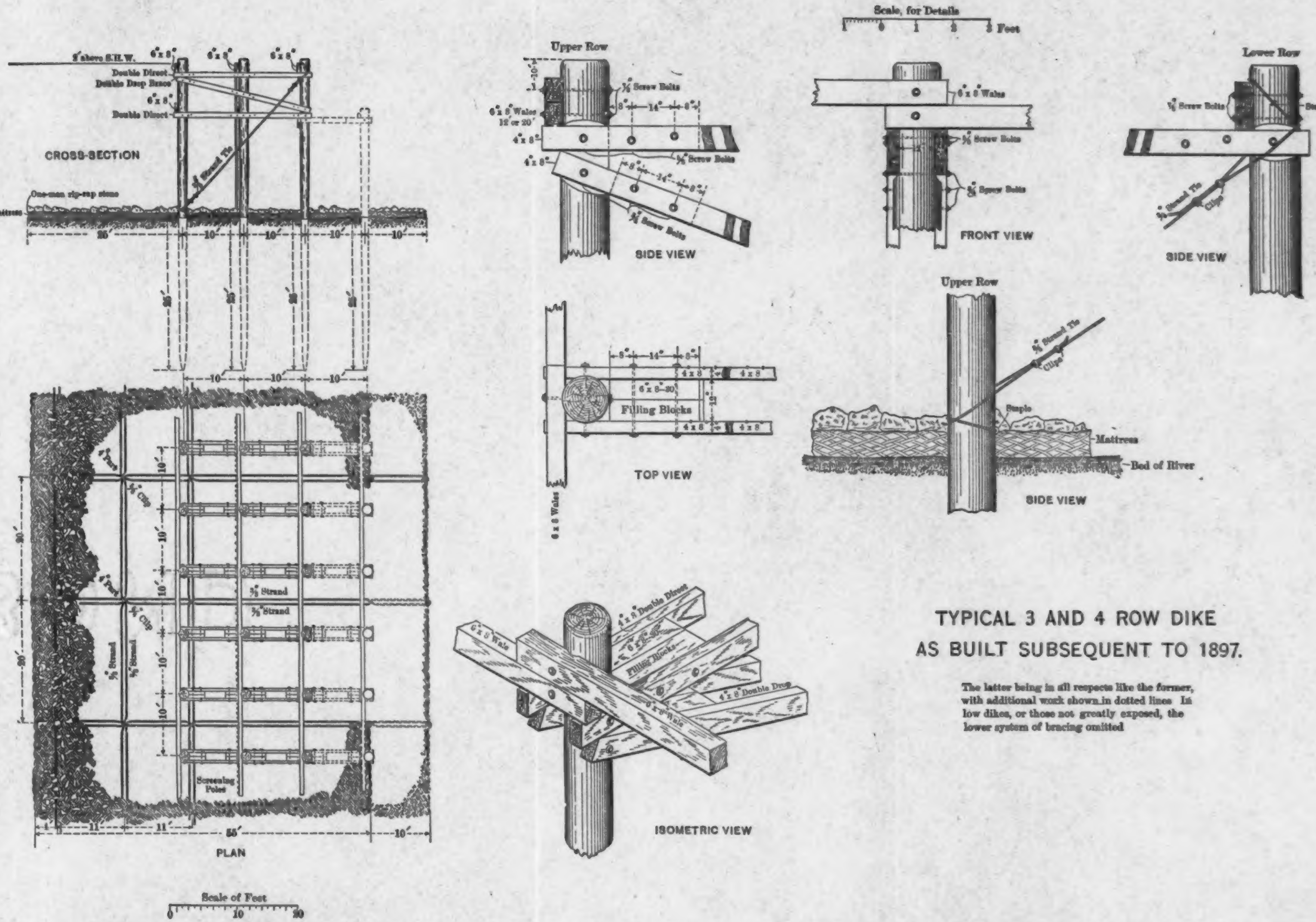
Fig. 4 shows the typical 3- and 4- row dike, as built during the period from 1891 to 1897.

After the foregoing specifications were formulated, three details which added to the strength and efficiency of pile dikes were adopted:

First.—All double, direct braces were provided with filling blocks, 30 in. long, fitted close up to the pile at each end of the brace, and held in place by two ¾-in. screw-bolts. The object of this device was to relieve or reinforce the bolts which fasten the brace to the pile.

Second.—Wire-strand ties were used, to transmit stress from the top of a dike in the lower row to the base of the structure at the upper row, thus adding a measure of stability previously obtained by the more expensive method of double-system bracing, or an additional row of piles. The ties are of several parts of ⅜-in. strand, or, one or more parts of ¾-in. strand, as indicated by the stress. They are usually attached to the upper pile before driving, at a point which when driven will be on or near the bottom, a round turn being taken on the pile, and the bight of it being fastened there by a staple; if it be a single-part tie, the short end is clipped on the other, close up to the pile, and the tie, whether of one or more parts, is then lashed up alongside of the pile until the foot-mattress shall have been sunk in place, when it is made fast at the top of the pile in the lower row. Several devices were used to obtain proper tension on the tie when in place. If composed of two or more parts, they were pin-twisted, after being made fast, until as taut as desired, and then the pin was lashed and left on the tie. One of the simplest and best methods for making one-part ties taut is, first to make the tie fast with as little slack as possible without the use of

PLATE XXVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 995.
FOX ON
MISSOURI RIVER IMPROVEMENTS.



TYPICAL 3 AND 4 ROW DIKE AS BUILT SUBSEQUENT TO 1897.

The latter being in all respects like the former, with additional work shown in dotted lines. In low dikes, or those not greatly exposed, the lower system of bracing omitted.

tools, then, with a spreader, composed of a jack and block placed on the direct brace, spread apart the tie pile and the one next above it in the same bent, until the tie is under the desired tension, then place the filling block in the down-stream end of the direct brace close up against the tie pile, and, finally, bolt the brace to the tie pile before removing the spreader.

Third.—The spacing of the poles forming the curtains of cross-dikes was given wider range, and, except in special cases, was made to increase uniformly and rapidly from the bank out; those in the shore-end bent space were placed in juxtaposition, or, say, from 24 to 28 poles were used; in succeeding bent spaces the poles were spread apart increasingly, to the extent indicated by local conditions. In special cases, where it was desired to invite the flow to move quickly from one part of the structure to another, the former was more closely curtained than the latter, or, the curtain was omitted on the latter part. Plate XXVIII shows the general plan and details of 3- and 4- row pile dikes as built subsequent to 1897.

Numerous other bracing devices were tried, with a view of economizing in material and cost of building, without impairing the strength and efficiency of dikes, but without marked success.

Miniature bents of dike work—about one-third size—of various forms, old and new, were erected on shore, and subjected to known stress until broken or overturned. The most interesting of the developments from these experiments was the apparent utility of a ground-brace, *i. e.*, a direct transversal at the river bottom. Such braces were introduced on a number of dikes in the upper reaches, but no special report as to their efficiency was made; and the difficulty and cost of placing them led to their abandonment. The experiments clearly demonstrated the superior efficiency of a simple division of the bent quadrilateral into two triangles with double, direct braces in which filling blocks were used, and diagonal ties of galvanized strand. Additional members involved ambiguity of stress, owing to the character of the construction.

There are two things which mar the efficiency of pile cross-dikes:

First.—Immense quantities of drift-wood, borne on the surface and at all depths during flood stages, find lodgment to a greater or less extent in unevenly distributed masses upon the structures. This

often results in breaching the dike, by overturning it, or, by crushing the structural parts; scour, introduced by concentrated flow, or excessive head, may be a primary, or, contributory cause, and, of course, once a breach is formed it rapidly widens, due to scour, until head has been dissipated to such an extent that velocities are reduced below that at which scour can take place. Even though the structure be not breached, its curtain may be rendered practically inoperative by drift-wood, so that the flow through the dike is very uneven and the deposits formed are correspondingly irregular.

Second.—There is always more or less of a pot-hole or trench at the stream end of the dike, which attracts the flow and prevents the formation or maintenance of deposits quite out to the end of the structure; more or less eddy action is in persistent attendance, and, as the structure deteriorates with age, constant exposure to the forces of the river is more and more likely to destroy it; and, once the outer end of a dike is destroyed, the remaining portion yields more readily. Scour at the stream ends of dikes is first caused by increased velocity due to release of head on the structure. The pot-hole or trench is formed and maintained as the result of scour in the presence of a fixed object of limited extent; and the form, area and depth of the pot-hole depend upon the form and extent of the structure, the head due to the resistance it offers to flow, and the character of the river bed at that place.

Initial scour having occurred, flow is attracted, and, because of the fixed object (the outer end of the dike), velocity and, therefore, scouring capacity are increased. Increase in depth and velocity is followed by decrease of local width. A bar, or reef, over which the attracted flow pours into the deepened section, moves down toward the structure, increasing the scouring effect and continuing to advance until its lower face is swept by a current strong enough to carry away, as fast as contributed, the material brought in over its crest. The pot-hole or trench thus formed is defined on one side by the bar, and on the other by the structure, and it conforms somewhat to the latter. The trench extends but a short distance beyond the structure, being most pronounced in front of it, or slightly below the point of strongest impact against the structure. Easement of flow is found immediately below the dike, and the slackened cur-

rent being unable to carry the materials scoured out from, or brought through, the trench, drops them, forming a bar; and the tendency is for the flow to divide on the bar, forming two waterways. The location, height and extent of the bar, and the predominance of one or the other of the two waterways depend largely upon the form and size of the structure, but vary also with stage, approach and other elements of flow.

It will be seen, therefore, that in reaches of river requiring advancement of both banks, the use of cross-dikes will result in a more or less divided flow; the cross-section of a reach so treated will be characterized by shallow mid-stream depths and relatively greater depths along both banks. In straight reaches, the flow may approximate an equal division along each bank; in curved reaches, up to the limits of curvature when chording effect is operative, the flow would be increasingly on the concave side.

Several devices have been tried with a view of remedying or ameliorating these effects, but with only a small measure of success.

In addition to the dike heads previously mentioned—trails and T-shaped heads of pilework similar to the main structure, and pile-cluster heads—short, subaqueous spurs of rip-rap, and curved heads of pilework have been tried.

The stone spurs referred to were built at the outer ends of the upper two dikes of a group in First Reach.

The result, after several seasons' exposure, was the formation of two pot-holes at each dike, one above and one below the spur, connected by a trench around the end of the spur. Each of the new pot-holes, however, was of smaller area and less depth than the original one, and the trench was not sharply defined. There was a gradual degrading of the spurs, apparently due in part to scour along their up-stream sides.

Two dikes with curved ends, of pilework, were built in the vicinity of Nebraska City. It was expected that, in such form, the flow about their ends would be eased, making less eddy and causing the accretions to form nearer to the channel lines; also that heavy drift-wood would not accumulate against them. They were designated and are hereinafter referred to as Dikes Nos. 3 and 5. Both are located on the left bank, opposite Nebraska City, Neb., Dike No. 3 being 1 425 ft. above Dike No. 5. Both structures are at

times subjected to severe attacks by the river, Dike No. 5 oftener and more severely than Dike No. 3.

Dike No. 3 was built in November, 1897, and is an extension to Stub-dike No. 3, constructed in May, 1897. It is in the form of a circular arc, 400 ft. in length, with a radius of about 340 ft. It is composed of 3-row work, except the lower 100 ft., which is 2-row work.

Dike No. 5 was originally built during March and April, 1898. It was formed of 80 pile-bents of 2-row work, 37 of which extended from the shore about normal to the flow; the remaining 43 bents formed the curved trail, made as a circular arc of 340 ft. radius.

Soon after the completion of Dike No. 5, there were several sharp rises, which caused extensive accretions to form under both dikes. Both structures were practically free from drift-wood.

Dike No. 3 is still intact, and has caught but little drift-wood; but the accretions formed by it have not been maintained, and are deficient.

During the June rise of 1899, the outer end of Dike No. 5, including practically all the curved portion, was carried away.

The rebuilding of a curved end, of new design, was commenced in November, 1899, but a combination of adverse causes—insufficiency of available plant and delays in receiving construction materials—held the work back until increased cost and loss of some of the work, due to severe weather conditions, forced a suspension of operations, with the structure incomplete.

The dike was completed in April, 1900, though not as designed, the foot-mattress being curtailed in width and length.

As rebuilt, it was composed of six concentric rows of piles, braced, provided with foot-mattress, and screened. The outer row of piles was on the arc of a circle of 800 ft. radius; the other rows were spaced radially inshore 10, 15, 15, 10 and 15 ft., respectively, between center lines, commencing with the outer row. The piles in the outer row were spaced at 10-ft. centers, and those in the other rows fell on the radials therefrom, except in the third and fourth rows, where piles were placed only on every fourth radial.

The outer row passed through Bent 44 of the original structure, and was 660 ft. in length, its up-stream end being 80 ft., on the circular arc, above Bent 44, and its lower end lying at the point of

tangency of the 800-ft. circle to the proposed line of rectification. The grade of the two inner rows—735 and 750 ft. radius—was level and at standard high water; the other four rows were built to a level grade 5.66 ft. lower, forming a flat level berm, 50 ft. in width throughout and 3 ft. above standard low water. Ten bents of 2-row pilework, with 10-ft. spacing, were built to a level grade, at standard high water, on a radial line through the eighteenth bent above the lower end of the structure, extending 108 ft. inshore from the 735-ft. curve.

Below Bent 43 of the old dike, the foot-mattress was suspended from the piles in the 735-ft. curve at standard low water. It was intended that the foot-mattress should have a total width of 155 ft., extending from 15 ft. inside of the inner row of piles to 75 ft. beyond the outer row; and it was made thus for about one-half the length of the curved head from the up-stream end. Below that, to the fourteenth bent from the lower end of the structure, the width of mattress outside of the outer row was curtailed, first to 60 ft., then to 45 ft.; and below the fourteenth bent the mattress in front of the dike from the lower end was omitted, the outer row of piling being just inside the selvage edge of the mattress.

It was expected that the effect of the wide low berm in the structure would be to correct in a large measure the trenching effect of the flow past the dike, but, owing partly to the curtailment of the foot-mattress, and principally to the fact that the berm flow was obstructed by drift-wood which found lodgement in the open work of the berm, little benefit was realized from it.

The object in building the 2-row radial dike, near the lower end of the curved head, was to shut off flow along the inner row of piles. It was prompted by the results obtained with the original structure, one of which was a persistent flow in a narrow chute just inside its inner row of piles.

Early in June, after the dike was completed as above, it was subjected to a specially severe attack, and on the 10th of the month thirteen bents of the high work—750 and 735-ft. circles—in the upper half of the curved head, gave way and were destroyed. The berm was not damaged.

In the fall of 1901, the breach in the high work was closed, the structure was reinforced by additional bracing and re-curtained

throughout; drift-wood was removed from the berm; and, to hasten the closing of the waterway behind the dike, a length of 300 lin. ft. of standard, deep-water abattis was built in one piece, extending from the in-shore end of the radial 2-row pilework to the accreted foreshore of the main bank, on a line parallel with the stem of the dike.

The first effect of the abattis was to cause a general raising of the bottom, but, later, a threatened flanking at the shore end necessitated the use of a large number of fascines.

When last inspected, Dike No. 5 appeared to be intact, but the berm, for a considerable length at its upper end, was covered with a compact mass of heavy drift-wood nearly to the level of standard high water; and there was still a flow through the structure in the locality of the repaired breach.

Longitudinal Dikes, With and Without Stems.—Though none of these structures was built in the stretch of systematically improved river in First Reach, the results obtained by them, elsewhere in the river, are thought to be of such importance and promise as to warrant a description of them here.

The first longitudinal pile dike on the Missouri River was built by the Commission in 1895-96 for the purpose of masking a pronounced bay in the shore line of the left bank above the Interstate Bridge, above Omaha, Neb. As far as completed, it was a marked success, promptly accomplishing and maintaining, without repair or additional attention of any kind, the object for which it was built. It conformed to the proposed line of rectified shore, and connected with the outer ends of a series of cross-dikes built from the main bank on lines about normal to the flow.

The piles were placed in triangular plan, adjacent piles measuring in place 10 ft. between centers, thus forming (longitudinally) three rows, each about $8\frac{2}{3}$ ft. between center lines. The piles were capped with 6 by 8-in. timbers of long-leaf yellow pine, drift-bolted thereto; the usual wales, direct braces and drop-braces were attached with $\frac{7}{8}$-in. screw-bolts. The foot-mattress was of the woven form, 125 ft. in width, 100 ft. of it being outside of the outside row of piles.

The longitudinal or training dike is 2 600 ft. in length, and, at its up-stream end, is connected with the main bank by a cross-dike.

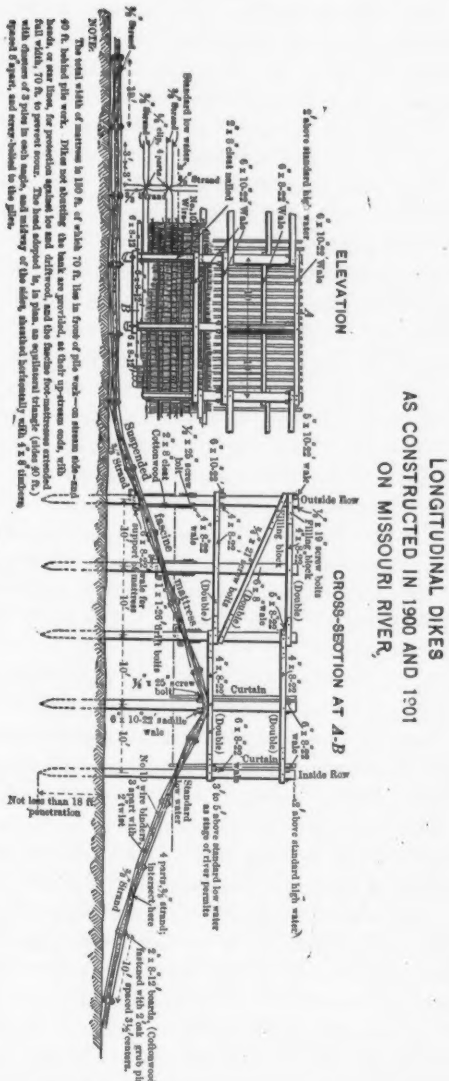

LONGITUDINAL DIKES
AS CONSTRUCTED IN 1900 AND 1901
ON MISSOURI RIVER,
ELEVATION
CROSS-SECTION AT A-B

FIG. 5.

Owing to deep water on the line of the latter, a woven mattress, suspended to the piles, was used instead of the usual pole curtain.

The longitudinal dike, for a length of 750 ft. from its up-stream end, and in addition to the usual sill or foot-mattress, was provided with a woven mattress, one edge of which was suspended to the piles and the other edge woven into the sill mattress, so that, in position, it formed a slope in front of the dike. It was expected that the space underneath the suspended mattress would be filled by deposit, thus forming a revetted sub-bank.

During the fiscal year of 1901, four longitudinal dikes of new design, as shown in Fig. 5, were built: one each in the vicinities of Nebraska City and St. Joseph, one in Wilhoite Bend, above Glasgow, and one in Howard Bend, above St. Charles.

The first two abutted the bank; the other two were connected with the bank by a single stem or cross-dike, also of new design, run out normal to the flow. See Fig. 2, Plate XXV.

The design for these dikes, which was closely followed in the essential features, provides for a 5-row pile structure, built to a level grade 2 ft. above standard high water. The piles are spaced at 10-ft. centers in the rows, and the rows are 10 ft. between center lines. The system of bracing is practically the same as that adopted for standard double-system bracing on cross-dikes, except that: strand ties are omitted; the wales on the first or stream row of piles are 8 by 10 in., instead of 6 by 8 in., and an additional wale is attached below the lower system of bracing; smaller reinforcing wales are attached immediately back of the lower two wales, on the down-stream side of the pile; wales on the second row of piles are omitted, except a single string of 6 by 8-in. midway of the upper and lower system; double, direct drop-braces from the upper system on the first row to the lower system on the third row are used.

The distinctive feature of the design is the suspended foot-mattress, with unusually wide fascine. It has a total width of 150 ft., 70 ft. of it being in front of, or on the stream side of the structure. Facine mattress was adopted because, though more expensive than the ordinary woven mattress as usually made, it offered better protection against scour through it, and greater strength. It is formed of compact willow brush fascines, about 8 in. in diameter, and of length equal to the width of the mattress—150 ft. The

fascines are bound and attached to each other, at intervals of 3 ft., by two continuous No. 10 galvanized wires, the latter being brought together between the fascines and twisted for a length of about 2 in. In this way, the wires were locked so that they would not render in the event of rupture, and an opening in the mattress was provided through which, it was thought, sufficient flow would pass to insure the desired fill under the mattress. The mattress is strengthened, longitudinally and transversely, by $\frac{3}{8}$-in. galvanized wire strands at top and bottom. These are put on taut, and fastened together at all intersections by cable clips embracing all four strands. The transverse strands are 10 ft. apart, and pass through the pile-work on the up-stream side of the piles. There are thirteen longitudinals, spaced as follows: 1, 10, 30, 50 and 60 ft., respectively, from the outer, or stream, edge of the mattress; on the in-shore side of the first, second and third rows of piles; on each side of the fourth or saddle row; on the stream side of the fifth row; and 20 ft. and 4 ft., respectively, from the inner edge of the mattress. To give the mattress additional longitudinal strength, and to hold the fascines together after the wires and wire strands have rusted out, lines of top- and bottom- board binders, spaced 10 ft. apart, are provided. These are 2 by 8-in. rough, cottonwood boards, 12 ft. long, fastened to each other through the mattress at intervals of $3\frac{1}{2}$ ft. by 2-in. oak pins and wedges.

The mattress is supported at the fourth row of piles on two 6 by 10-in. wales which embrace and are screw-bolted to the piles. At the third, second and first rows, it rests on 6 by 8-in. wales, which are supported by 1-in. square, iron-elbow, drift-bolts. As these bolts are to be submerged, they are driven into the piles at that stage of the driving of the latter when the designated point on the pile has arrived at the bottom of the leads; the pile is then driven to grade. The 6 by 8-in. supports are forced on the elbow-bolts and held there, pending the construction and sinking of the foot-mattress, by board cleats, nailed above water to the piles. The two 6 by 10-in. saddle wales—on the fourth row—are placed so that the surface of the mattress over them will be at an elevation from 3 to 5 ft. above standard low water, as the stage of river at the time may permit; and the supports at the other rows are placed on a grade sloping 1 on $3\frac{1}{2}$ from the saddle wales.

The specifications provide that rip-rap, about 1 ton per lin. ft., shall be used to sink the mattress, the major portion being thrown on and near the outer edge, reserving for the supported portion of the mattress only enough to overcome, with some margin, its buoyant effort. As the mattress inshore from the fourth row serves principally to catch overpour, it is allowed to assume a natural position, with little or no ballast.

The usual pole curtains, with somewhat wider spacing than for cross-dikes, are attached to the fourth and fifth rows. It was expected that the space under the suspended mattress would be quickly filled by deposit, thus giving in effect a revetted and, therefore, permanent sub-bank with stable slope; and the upper bank would be advanced, by deposit after one or more floods, through the agency of the curtains, out to the line of the dike. It would only be necessary then to pave and spawl the upper bank, as provided for in standard bank revetment work, to have a new fixed bank of desired height, location and alignment; and that part of the dike above water could be removed, if desired.

Dike-heads.—Longitudinal dikes which do not abut the bank are provided with heads, or starlings, at the up-stream ends, for protection against ice and drift-wood; and the foot-mattress is extended to full width, 70 ft., to prevent scour. In plan, the head adopted is an equilateral triangle with 40-ft. sides, with clusters of three piles in each angle and midway of the sides, sheathed horizontally with 4 by 8-in. timbers spaced 8 in. apart and screw-bolted to the piles.

Stem-dikes.—The adopted design of stem-dike for connecting such structures to the bank contemplated a 3-row pile structure, with 10-ft. spacing, built to the same grade as the longitudinal dike, and on a line about normal to the flow.

The foot-mattress is of the fascine form—the same as used for standard longitudinal dikes—board binders being omitted. It is 90 ft. in width, 40 ft. of it being on the up-stream side of the pile-work and 30 ft. below the lower row of piles. The mattress is suspended from the middle row of piles on two 6 by 10-in. continuous wales which embrace and are screw-bolted to the piles at the same elevation as those in the longitudinal dike.

The usual wales, and double, direct braces with filling blocks, are

attached at the tops of the piles; double, direct braces extend from the upper to the middle row of piles, at the same elevation as the saddle wales; and a string of wales is attached to the upper row of piles immediately above the latter direct braces.

A strand-tie, $\frac{3}{4}$ in. in diameter, is attached to each bent, from the top of the pile in the third row to the pile in the first row at the bottom direct brace.

The original design provided the usual pole curtain on the middle row, the lower ends of the poles terminating 2 or 3 ft. below the saddle rows.

While the results obtained with longitudinal dikes of the form just described have been satisfactory, as far as their effects upon the conditions of flow are concerned, their cost is so great that it is not probable they will be used in that form, except perhaps to a limited extent in special cases.

The results from the stem-dikes were also satisfactory. It was found necessary, however, on account of scour through the foot-mattress, to extend the curtain on the middle row to the bottom of the river, and to attach another similar curtain to the lower row. It was also found that a foot-mattress 40 ft. wide above a stem-dike is not sufficient. The up-stream edge of the foot-mattress of the stem-dike in Howard's Bend, when in position on the bottom, some 20 ft. below standard low water, was only 22 ft. above the upper row of piles, and the structure was in imminent danger of loss due to scour, or overturning. The width of mattress on the surface should vary with the depth, so that, when sunk in place on the bottom, its up-stream edge would be not less than 50 ft. above the dike. For 25 ft. below the up-stream edge, the mattress should be made as tight as possible, to avoid scour through it; and beyond that it should be opened, somewhat, to permit flow, which will favor deposit under the mattress.

Sheer-dike.—A sheer-dike was built above the dikes in Howard's Bend, on the line of the middle row of the longitudinal dike produced, and extending up stream from the starling or dike-head to the main bank. It was composed of nine clusters of four piles each, driven to a level grade, 5 ft. above standard low water, and spaced 25 ft. apart, between centers of clusters. The clusters were served at their tops with $\frac{3}{8}$-in. strand, embracing all the piles in the cluster.

The object in view was the protection of the foot-mattress of the stem-dike from ice and drift-wood, and, incidentally, to take up some of the head, pending the formation of deposits under the mattress.

Pile Sinking.—Prior to 1892 pile sinking on the Missouri River was done almost exclusively by the hydraulic jet. Thereafter, Nasmyth steam hammers, of Cram and Vulcan patterns, sizes B and No. 2, respectively, were used more and more frequently, to the final exclusion of the jet apparatus, after several seasons' work.

From the first crude outfits, consisting of a single set of pile-leads mounted on a small boat, with a hand force-pump for jet supply, more elaborate plants were developed, resulting, finally, in the use of two hulls, one 19 by 76 ft., called the pump-boat, the other 19 by 48 ft., called the cross-boat.

The pump-boat was provided with a steam pump, having a capacity of about 250 gal. per min., for jet supply; a boiler and accessories, and two steam capstans. A cabin covered all except the capstans, and provided sleeping quarters for the crew.

The cross-boat was provided with six sets of pile-leads—three sets on each side, spaced at 10-ft. centers—built in the form of a tower, 48 ft. high, and having three working platforms, 15, 30 and 40 ft., respectively, above deck. A steam hoist and steam capstan, also, were on the cross-boat. The former was used in raising the piling and jet-piping attachments into position, and in pulling down on the pile while sinking; the capstan was used to adjust the boat in position. Steam for operating the machinery on the cross-boat was furnished from the boiler on the pump-boat through "gooseneck" pipe connections. The water for the jet supply was carried from the pump-boat to the cross-boat through a 4-in. rubber hose, then along deck to near the center of the base of the tower, in 4-in. iron piping, and thence vertically to the second working platform, where the size of the piping was reduced to 2½ in. and it was connected with a rubber hose of that size, in the axis of a compensating hose reel. The latter was arranged with variable counterweights so that only the necessary length of hose required to reach from the reel to the jet-pipe on the pile was off the reel at any time; and when the jet-pipe was removed from the pile after sinking, the hose was rewound upon the reel automatically. A valve at the pump released the pressure when in excess of 100 lb.

In operating, the two boats were fastened together firmly, the cross-boat lying at right angle to the pump-boat at the pump end of the latter; so that, when in position for working, the pump-boat was on the line of the up-stream side of the upper row of piles in a dike, and the three sets of leads on the upper gunwale of the cross-boat were on the center lines of the upper, middle and lower rows of piles.

Exhaustive experiments were made with nozzles of various forms and sizes, applied in many different ways at and near the base of the pile; and, except in bed formations of gravel or indurated clays, the results, in the main, were satisfactory.

With a jet, it is impossible to sink piles beyond a limited depth in gravel, the return flow being dissipated through it. The prevalence of gravel in the lower valley led to a trial of steam hammers, and they proved to be so much more reliable and efficient, in every way and under all conditions, that, as above stated, they finally superseded the jet sinkers.

In the steam-hammer outfit, the cross-boat was dispensed with; and the hammer operated in a set of special leads, 45 to 52 ft. in height, mounted on the pump end of the pump-boat. The steam pump was replaced with a steam drum-hoist.

In operation, the driver was generally kept head on, to the current, though, when necessary, it could be placed across the current, as the hull was side-raked. Ordinarily, driving progressed from the shore out, each bent being driven complete before removing to the next one. A 100-ft. barge, carrying the supply of piling, was kept alongside the driver on the inshore side and moored to the piles; it served to steady the driver in adjustments for position, as well as during the operation of driving.

The crew of the steam hammer outfit was composed of a foreman at from \$60 to \$70 per month, a steam engineer at \$90 per month, a fireman at \$35 per month, and six laborers at \$1.10 per day. All were subsisted by the Government. The total daily expense of the driver, including labor, subsistence, fuel and engineer's supplies, was about \$20. The average day's work—one day with another throughout say a month—was about 20 piles, driven to a penetration of 25 ft. With all conditions favorable—a skilled crew, medium depth and current, a bed of sand, absence of wind, ample supply of

piling of proper lengths, straight work and no breakage, 80 piles per day have been driven.

For land driving, the leads and hoist were taken ashore on skids and rollers, or, on a frame with two axles and grooved wheels, the latter running on 8 or 10-in. iron piping. Steam was piped ashore from the boat's boiler.

Standard Revetment.

In a previous part of this paper the various forms of revetment used during the several stages in the development of what has been adopted and is now known as standard revetment were described. The following is a description of the latter:

The natural sequence in which the several branches of revetment construction are carried on is: Bank grading, mattress weaving and anchoring, sinking and ballasting mattress, spawling inshore edge of mattress, paving upper bank, and covering pavement with spawls.

Bank Grading.—The upper bank is graded, between the water surface existing at the time and the 2-ft. contour above standard high water, to a slope varying from 1 on 2 to 1 on 3½, or flatter in special cases, depending upon the character of the material composing the bank and the attack to which it is exposed. The grading is done with a hydraulic jet. As far as practicable, the irregularities in bank lines are faired out in grading; prominent points are cut back, leaving a low, flat berm from which the slope to the top of the grade is about an average one. In former practice, such points were graded to a uniform slope, flatter than the average. Care is taken that the graded slope shall be smooth and free from ruts or gulleys, and it is of great importance, also, that it shall, throughout, be of material *in situ, i. e.,* no part of the slope above water should be formed of tailings. The settlement of tailings leaves a shoulder in the bank which, unnoticed or not properly treated, invites destruction of the upper bank work. Partly on this account, the best revetment can be constructed when the stage of river is lowest; and all revetments built at a stage of 5 ft. or more above standard low water should be inspected at a lower stage, with a view of discovering and treating shoulders in the bank which are more than 3 ft. above standard low water.

FIG. 1.—HYDRAULIC GRADER AND PARTY, GRADING BANK FOR REVETMENT.

FIG. 2.—PARTY AT WORK, WEAVING MATTRESS.

Grading Outfit.—The hydraulic grading outfit, as most recently provided, consists of: A grading boat, Fig. 1, Plate XXIX, having a hull 20 by 80 ft., on which there is a compound, condensing steam pump, of the packed-plunger pattern, having a capacity of about 600 gal. per min. at a piston speed of 100 ft. per min.; a steam capstan at the pump end of the boat for use in moving the boat and for light snag pulling or grubbing; a hand capstan at the other end of the boat; and a boiler for steam supply. The pump and boiler are inclosed with a cabin which provides sleeping quarters for the crew. A mast at each corner of the pump end of the cabin is used to operate a boom in which the discharge hose is carried ashore on either bank.

The water supply for the pump comes through a screened suction pipe in a screened well within the hull. The discharge is carried in 6-in. iron pipe from the pump to both gunwales, at the foot of the masts, for use on either bank of the river, the one not in use being capped. From the boat to the play-pipe, ashore, the water is conducted in 4-in. rubber hose, a length of 75 ft. being required under ordinary conditions. The play-pipe at the shore end of the hose is of copper, 4 ft. in length, and tapers from 4 in. in diameter at the hose to 2 in. at the nozzle end. It is operated by a wooden lever, about 8 ft. long, to which it is fitted and strapped. The lever is held against recoil, and is given full movement in all directions by means of a gimbal, which sets, at an elevation of 16 to 20 in. above grade, in a 2-in. gas pipe driven firmly into the earth. The jet is played upon the bank at close range, the loss of time involved in the necessary resettings being repaid by the greater efficiency of the jet. The form of nozzle tip which gives the best results—the most compact, solid jet—is cylindrical for 2½ diameters of the orifice back from the orifice, and thence, to the play-pipe connection, conical. The cylindrical surface of the tip should be kept highly polished. With the pump working at full capacity, and an average bank, a jet 1¼ in. in diameter gives the best results.

A grading crew consists of a steam engineer at $90, a fireman at $35, a nozzleman at $50, and two laborers at $35 each per month; all with subsistence. The total daily expense for labor, subsistence, fuel and engineer's supplies is about $15. In a day's work, from

500 to 1 800 cu. yd. of earth are removed, the amount depending upon the character of the bank and the slope to which it is graded.

Mattress Weaving and Anchoring.—The mattress is of a continuous, woven-willow construction, extending from the 3-ft. contour above standard low water on the graded slope, or, from the water's edge if the stage be above that contour, to 80 ft. beyond the standard low-water contour, thus providing for a mimimum width of mattress of about 86 ft. It is compactly woven to a thickness of not less than 12 in. or more than 18 in., according to the size of the brush or according to the exposure. Dense thicket-grown willows, which average about 1½ in. at the butt ends, are best adapted to the work, with smaller brush—from ½ to ¾ in.—for weaving a selvage on the shore and stream edges of the mattress. Brush having a larger butt diameter than 2½ in. should not be used, and if much of the brush is greater than 1½ in. in diameter, enough small brush should be used with it to give the requisite compactness. The strength of a woven mattress is no greater than the frictional resistance to motion of the composing brush. To give additional strength, required to meet the severe strains imposed while the mattress is being sunk, as well as in subsequent movements or adjustments to scour, and, for the purpose of anchoring the mattress to the top of the bank so that it will not, under any conditions, slip or slide, a system of ⅜-in. galvanized-wire strands, running lengthwise and crosswise of the mattress, is used. Both longitudinal and transverse members are composed of two ⅜-in. strands each; one of which lies wholly underneath and the other wholly on top of the mattress—one part immediately under the other. There are six continuous longitudinal members, laid parallel, and spaced as follows, measuring from the outer selvage edge inshore: 6, 16, 31, 46, 66, and 85 ft., the last one being in the inner selvage edge. All these longitudinal members are paid out under tension, as the mattress is made, from reels on the mattress-boat, the top strand of each longitudinal passing through a fair-leader suspended some 18 ft. above the brush platform.

There are six transverse members (of two ⅜-in. strands each) to every 100 lin. ft. of mattress, extending, one strand on top of the mattress and the other directly underneath it, from the outer selvage edge to deadmen on top of the bank, back of the graded slope.

FIG. 1.—BANK REVETMENT. PARTY PAVING UPPER BANK.

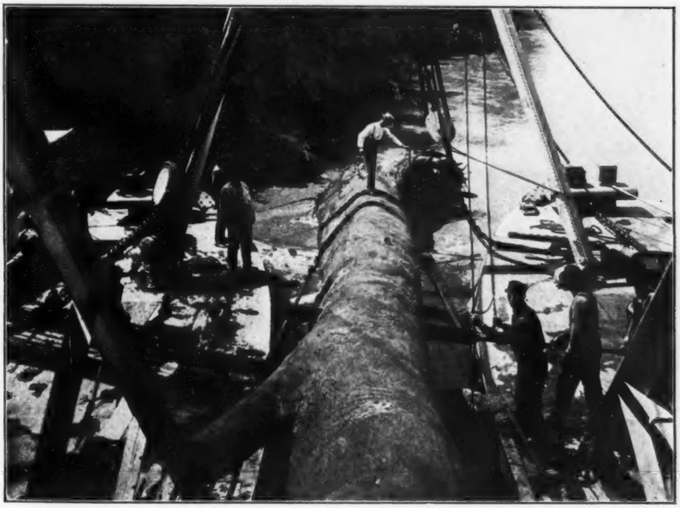


FIG. 2.—U. S. SNAG-BOAT "C. R. SUTER," REMOVING SNAGS.

They are laid about normal to the midwidth line of the mattress, so that, if the latter be a straight line, the transverse members will lie parallel to each other across the width of the mattress and $16\frac{2}{3}$ ft. apart, or, if curved, approximately on radial lines $16\frac{2}{3}$ ft. apart at midwidth of the mattress. This is readily accomplished, as the mattress-boat advances down stream with the weaving, on lines normal to the midwidth line; and the transversals are run out from a reel on shore, the bottom part first, enough strand being pulled through, past the outer selvage of the mattress, to reach ashore when laid back on top of the mattress and thus form a complete transverse member with a bight at the outer selvage. Both parts of the transversals are laid, just after the weavers pass the line, so that they are not woven into the body of the mattress. At all points of intersection of transversals with longitudinals, the four parts of strand are brought together and fastened by stirrup-bolts or clips of $\frac{7}{16}$-in. iron; but, before the fastenings are made tight, both parts of the transversals are put under tension, from the outer selvage edge to deadmen ashore, by means of blocks and tackle. The deadmen are either rough blocks of stone, containing from $2\frac{1}{2}$ to $3\frac{1}{2}$ cu. ft., or, pile butts 12 in. or more in diameter and 4 ft. long. The stone blocks should be not less than 10 in. thick. Pile butts of upland-grown white oak are preferred, but long-leaf yellow pine is used extensively. The deadmen or anchors are planted 8 ft. back of the top of the graded slope, and from 3 to 5 ft. deep, according to the character of the ground. From the top of the slope to the deadman in place, the strands lie in a narrow trench dug for that purpose.

As above stated, the weaving and, indeed, all branches of the work progress down stream. The start is sometimes made with the mattress-boat lying face on to the bank, and a quarter circle of mattress is woven, the lower inside corner of the boat being held against the bank while the upper end of it swings out, as the mattress is woven, to a position normal to the bank. The usual method, however, is to start the mattress of the full width at the up-stream end of the graded slope, the upper edge being stiffened, against buckling, by a bolster, or fascine of brush, extending the full width of the mattress, and into which the weaving is meshed. In both methods the longitudinals are carried ashore on converging lines, leading well up stream to a substantial anchorage.

The boat upon which the mattress is made may be any decked hull of such length as will afford a flat working space of from 8 to 10 ft. at its outer end, and, with a short outrigger if necessary, a sloping deck from 10 to 15 ft. wide and as long as the width of the mattress. The weaving platform should slope about 1 on $3\frac{1}{2}$, and be provided with launching ways of 3 by 8-in. stuff, spaced from 6 to 10 ft. apart between centers. In the rear of the launching ways and on a level with their top ends, a platform, 10 ft. wide, or more, should extend the full length of the boat. The brush for weaving is brought in wire-bound bundles and laid crosswise—butts up stream—on this platform. The bundles are then opened and the brush is passed, one or more pieces at a time, to the weavers. The brush platform may be on a separate hull lashed to the lower side of the weaving boat. The wire strand is on reels mounted under the brush platform. Upright scantlings extend 20 ft. above the platform, for the support of the top longitudinal strands.

A crank capstan at each end of the mattress-boat serves to pull the boat down stream as the weaving progresses, the head lines, meanwhile, being slackened.

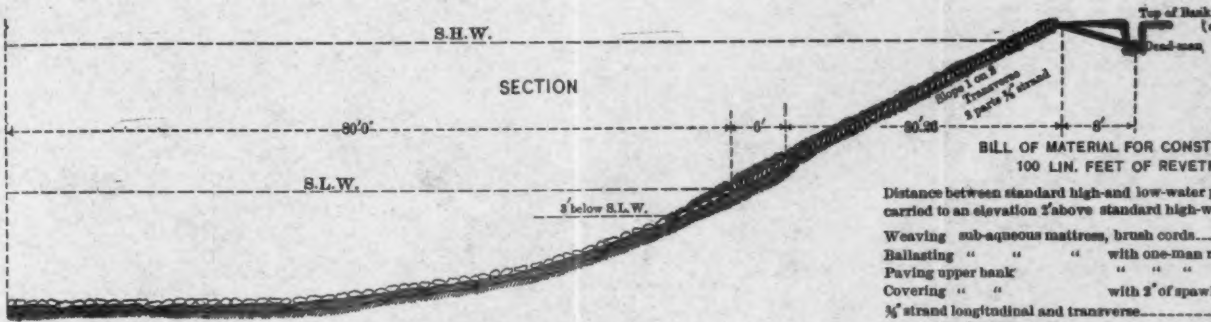
Specially designed mattress-boats, 25 by 70 ft., adapted for dike work as well as revetments, are self-contained. The lower gunwale is high; the upper one, low and raked for easement of dynamic head of flow, is flush at its shear line with the way floor; the latter is caulked, and rises on a slope of 1 on $3\frac{1}{2}$, giving a working surface $13\frac{1}{2}$ by 66 ft. (for revetment work 10 ft. of additional floor space is given at either end, as required, by detachable outriggers); the brush platform is 12 by 66 ft., its lower edge being flush with the plane of the lower gunwale; it is supported along the lower edge and on a line 8 ft. inboard by stanchions. The boat is stiffened by a longitudinal truss.

A weaving and anchoring force is composed of a foreman at \$60 per month, 11 weavers at $16\frac{2}{3}$ cents per hour, and 30 laborers at $13\frac{1}{3}$ cents per hour; all with subsistence. Fig. 2, Plate XXIX, is a photograph of a party weaving a mattress.

An average day's work is 100 lin. ft. of mattress, so that the average cost for labor and subsistence is between 65 and 70 cents per linear foot.

Sinking and Ballasting Mattress.—Specifications for standard

PLATE XXXI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 995.
FOX ON
MISSOURI RIVER IMPROVEMENTS.



NOTE.

Between limits of 3 ft. above and 3 ft. below S.L.W. spawls sufficient to fill interstices in mat and fair up selvage edge on the slope are distributed prior to paving.

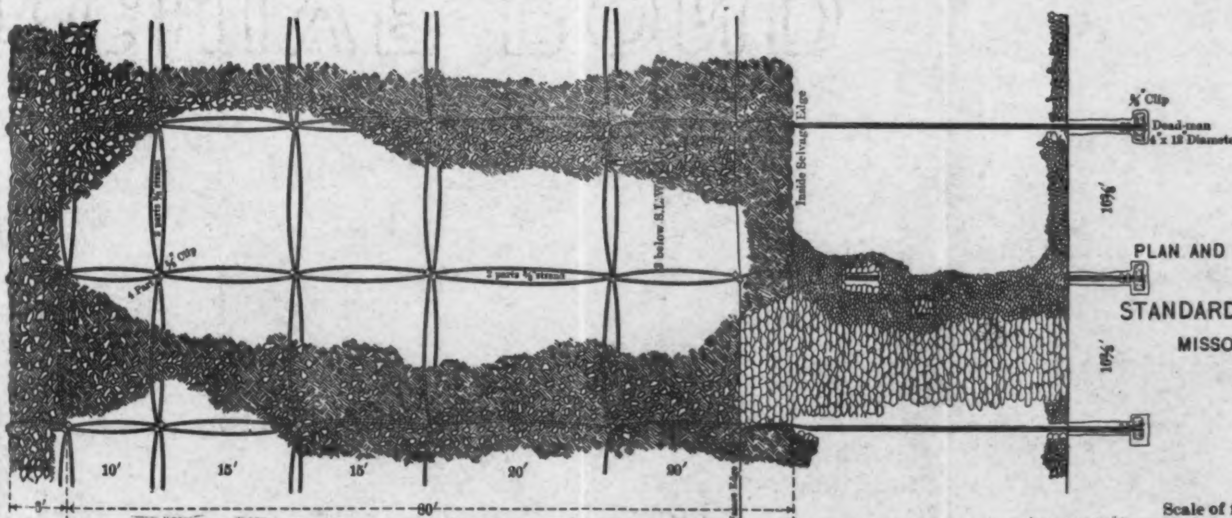
BILL OF MATERIAL FOR CONSTRUCTING
100 LIN. FEET OF REVETMENT.

Distance between standard high-and low-water planes is 16 ft., and paving carried to an elevation 2' above standard high-water plane.

Weaving sub-aqueous mattress, brush cords			60
Ballasting " " "	with one-man rock.	Cu. yds.	75
Paving upper bank	" " " "	" "	130
Covering " "	with 2" of spawls	" "	24
3/8" strand longitudinal and transverse		Lbs.	790
4 part clips for fastenings		No.	42
2 " " "		"	12
Dead-man pile heads 4 x 12 diam. long-leaf yellow pine or white oak, or, stone blocks, 32" x 12" x 12"		No.	6

NOTE.

Quantity of stone on upper bank varies with height between standard high- and low-water planes, the height of bank, the stage at which work is done, and with the grade of the slope, the paving being 12" thick at standard low-water and 12", 8" and 16" thick, respectively, at the top of 1 on 2, 1 on 2½ and 1 on 3 slopes, with 2" of spawls on top.



PLAN

PLAN AND CROSS-SECTION
OF
STANDARD REVETMENT,
MISSOURI RIVER

revetment provide that ¾ cu. yd. of rip-rap per lin. ft. of mattress shall be used in sinking it to close contact with the bottom, the distribution being such that the weight per square foot of mattress increases from the shore out. It is also specified that, in addition, 50 cu. yd. of stone shall be placed on 50 lin. ft. of mattress at the head, and 1 cu. yd. per lin. ft. on all laps. The stone is thrown from a barge, which is dropped down stream over the mattress, with its outer end somewhat in advance of the shore end. As the work is laid off in sections of 100 lin. ft., and the contents of the barge are known, the quantity of stone placed on each section is readily ascertained. It seldom happens that the full complement of stone is placed in sinking, a sudden shifting of the barge being often necessary to prevent buckling, especially in a swift current; it is usual, therefore, to go over it a second time. The sinking is not carried closer than from 150 to 200 ft. above the mattress-boat, and as much as 1 000 lin. ft., or even more, of mattress is sometimes made before sinking is commenced. There is danger in having so much mattress afloat, however, as a sudden rise may occur, and, though it might not otherwise damage the work, it would possibly foul the mattress with drift-wood so as to impair its efficiency seriously.

Spawling In-Shore Edge of Mattress.—This operation, which follows sinking and precedes paving, consists in filling well the interstices of the mattress, from its in-shore edge to the contour 3 ft. below standard low water, with small spawls or quarry chips; and, fairing up, with the same material, the shoulder formed by the edge of the mattress where it lies on the graded slope. The former is done to stop wave action through the mattress and to solidify the mattress against ice in the event of the removal of the paving or ballast; the latter to give a fair surface for the pavement, and to check wash from surface drainage. About 0.1 cu. yd. of spawls per lin. ft. is required.

Paving and Spawling Upper Bank.—The upper bank, from the contour 2 ft. above standard high water to standard low water, and as much lower as the existing stage of river permits, is covered with a paving of rip-rap, 12 in. thick at standard low-water contour and 12, 8 and 6 in. thick, respectively, at the top of 1 on 2, 1 on 2½ and 1 on 3 slopes. A 2-in. covering of spawls or fine quarry chips is put on the paving, thus completing the revetment. The paving is

done from the top of the slope down, the stones being set up edgewise and placed with some care to make a compact covering, so that when completed ready for the spawls it presents the appearance of a reverse shingling. In this form it is well adapted to resist wave action, and dislodgment by ice, drift-wood or other forces to which it is likely to be exposed. The spawls, thrown on at and near the top of the slope, are raked down over the pavement, and, after filling the interstices, find lodgment in the series of steps in the paving surface.

As will be seen from the foregoing, the quantity of rip-rap for paving, as well as the spawls required per linear foot of bank will vary with the height between the planes of standard high and low water, the height of the bank when lower than 2 ft. above standard high water, the stage at which the work is done, and, to some extent, with the grade of the slope; but the paving stone will average about 1½ cu. yd. per lin. ft., and the spawl covering will be less than ½ cu. yd. for a height of 16 ft. between the high- and low- water planes. Fig. 1, Plate XXX, is a photograph of a party at work paving the upper bank; and Plate XXXI is a plan and cross-section of the standard revetment.

Cost of Works on the Missouri River.—The cost of standard pile-dikes and revetments necessarily varies so widely that unit costs cannot be stated definitely. The following are the principal elements which enter the question of cost:

Difference in elevation of high- and low- water planes; height of bank; distance of work from base of supplies; plant charges; extent, and degree of concentration, of work; season of the year in which carried on; and conditions of flow. For preliminary estimates, however, \$10 per lin. ft. of cross-dike or revetment and \$35 and \$20, respectively, per lin. ft. of 5-row longitudinal and 3-row stem-dikes, as described herein, may be used to cover all items.

In carrying on numerous works at widely separated localities, it was found that the field charges against the allotment were distributed as follows:

For actual construction 67%; for care, repair and moving plant 22% (this includes an item of only 5% for light repairs); administration 9%; and 2% for all other items, including surveys and travel.

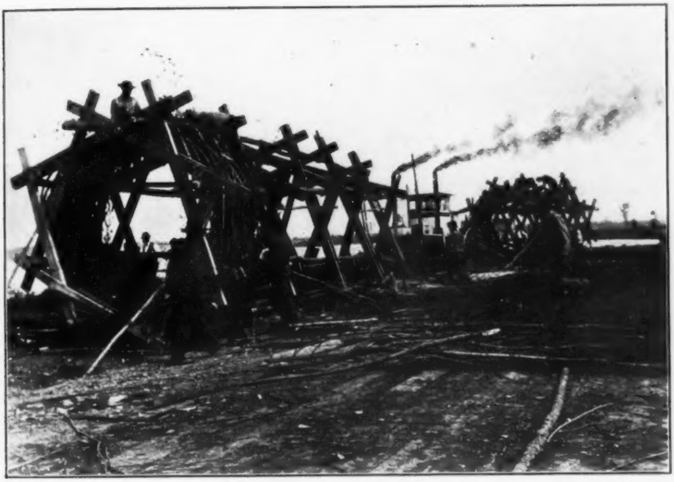


FIG. 1.—A GABION, IN PROCESS OF CONSTRUCTION.

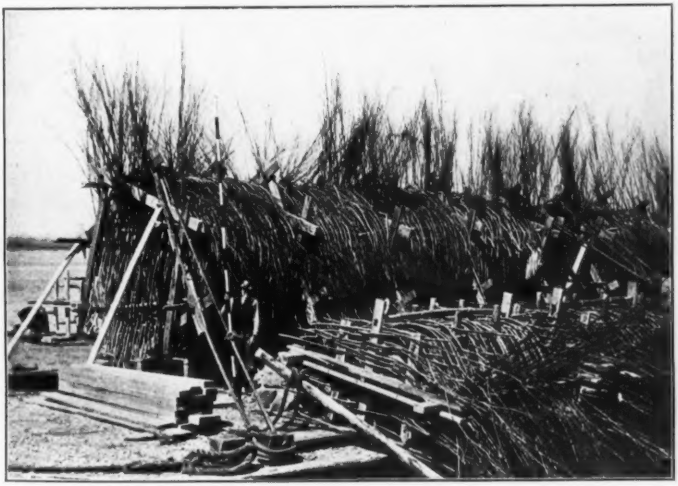


FIG. 2.—GABIONS AND BURRS, READY FOR USE.

SNAGGING.

Under general appropriations for the removal of snags from Western rivers by the General Government, beginning about 1856, ceasing during the Civil War, and continuing intermittently thereafter, the Missouri River, the "Snaggiest" of all streams, received its modicum of work for the avoidance of danger to its navigation from this cause.

The necessity for such work on the Missouri River is made apparent by the statement that the wrecks of some 300 steamboats now lie embedded in its sand. Of these wrecks, 195 were caused by snags and 11 by rocks.

The boats used in the removal of snags and other obstructions were crude and inefficient, in their earliest forms, but were developed into model machines for the work they had to perform. The hulls had been built of wood, and were of the catamaran type, up to 1874, when the first contract for iron construction, with a single hull and a forked bow, was entered into.

The *J. N. Macomb* (1874), the *H. G. Wright* (1880) and the *C. R. Suter* (1888), the latter two being steel, are still in service and highly efficient.

The dimensions of the largest boats are: Length 187 ft., beam 62 ft., and depth of hold 8 ft. Their draft is about 4 ft. The *C. R. Suter,* especially designed for the Missouri River, is 187 ft. long, 52 ft. beam, and 7 ft. hold, with a draft of 3 ft. Their present valuation is from $100 000 to $120 000 each.

These boats were built from designs by Colonel Charles R. Suter, M. Am. Soc. C. E., who was in charge of the river from 1874 to 1896, during the latter portion of the period—1884 to 1896—being President of the Missouri River Commission. To that able officer and to his able successor, Colonel Amos Stickney, is due the credit for much that is of scientific interest and value in our present knowledge of the physics of detrital rivers and methods for their control.

From March, 1889, when placed in commission, to June, 1902, the *C. R. Suter* destroyed 17 767 snags, removed 69 wrack-heaps, felled 6 076 trees likely to become snags, and traversed 18 108 miles of river. The total amount expended in the operations of the boat, including repairs, etc., was about $380 000.

Fig. 2, Plate XXX, is a view of the *C. R. Suter* removing a large snag.

Fig. 1, Plate XXXII, is a photograph of a gabion, in process of construction. Gabions are made of various sizes, from 8 to 12 ft. in diameter, and of any length up to 30 ft. They are used in building submerged groynes at bank-heads.

Fig. 2, Plate XXXII, is a photograph of some gabions and burrs ready for use.

DISCUSSION.

Samuel H. Yonge, M. Am. Soc. C. E. (by letter).—The writer congratulates Mr. Fox, his colleague for twenty-one years on Missouri River work, on his admirable paper describing the works for improving that stream, and believes that his efforts in bringing the subject to the attention of this Society will meet with its commendation. The part of the paper relating to later experimental work with "bank-heads," conducted on the lower reaches of the Missouri, principally after the writer had severed his connection with work on that river, is very interesting, although the experiment was interrupted before being concluded. Mr. Yonge.

The improvement of the Missouri is probably the most interesting river work ever undertaken, and there is no other river on which so many physical data have been collected—excepting the Lower Mississippi, which is, in fact, a lower reach of the Missouri—and such diverse and extensive experimentation conducted, with novel and original methods of improvement. There are few great rivers, if, indeed, there is another, in which various fluvial phenomena develop so rapidly as in the Missouri. Its regimen is in a condition of perpetual transition, and changes, for the consummation of which, in the majority of other non-tidal rivers, indefinitely long periods are required, in it are fully wrought within a few days. For this reason, it is one of the best subjects for the student of river physics and hydraulics, although, on account of the complexity of its phenomena, it is sometimes difficult to connect visible effects with their obscure causes.

When improvement work was begun by the General Government, in 1878, there was available practically no literature applicable to the subject, excepting accounts of experimental works of narrow scope on East Indian rivers; reports of the construction of several bridges crossing the river, relating principally to piers and foundations, and incidentally to the materials now filling the ancient channel; and of the limited experience of the General Government on the Mississippi in the vicinity of St. Louis, Mo.

During the early years of the improvement of the river, the large expenditures made by railroad and bridge companies for protecting their properties from the river's encroachments, by means of stone spurs and revetments, increased the river's bad reputation, and caused it to be generally regarded as incorrigible and unconquerable, except by using methods of "main strength." In one instance it is stated that a railroad company protected a stretch of bank, about 200 ft. long, contiguous to its track, by the daily expenditure of a train load of stone, for a period of three months

Mr. Yonge. (estimated at upward of 4 500 tons). This method—even had its adoption seemed advisable, which was not the case—was, on account of its cost, beyond the reach of those in charge of Government works, who, during the operations of the first few years, in consequence of the stint of appropriations made by Congress, had their ingenuity taxed to the utmost to construct works for protecting extended areas, with entirely inadequate means.

A knowledge of the failures and successes of Missouri River improvement works, outside of the localities where they were situated, is almost entirely confined to the few who were engaged in the undertaking, or those who have read the reports of the Engineer Officers and their assistants in charge of the improvements, published in the Reports of the Chief of Engineers, U. S. Army.

In the summer of 1879, the writer first introduced on Missouri River work, at Vermilion, S. Dak., the continuous woven mattress for protecting caving banks. The Vermilion mattress, 1 125 ft. long, was woven down the graded bank slope and over the channel, without a boat or supporting float, with a width of about 40 ft., and was then sunk with stone ballast.

As the plan of constructing woven mattress over the water without extraneous support did not give promise of the best results, a small flat, carrying light launching ways, was devised for constructing, in 1879, near Sioux City, Iowa, the second Missouri River mattress of the above type, which was about 2 000 ft. long, and was woven and sunk in one section, at a net cost of about $2 300. The use of the boat resulted in economizing both time and money, and in producing a better mattress. On account of the small appropriation for the Sioux City work, the mattress could be made only 40 ft. wide over the water and 30 ft. on the upper bank.

The superiority of the woven mattress, for protecting banks subject to sudden change of shape from scour, on account of its flexibility; for shielding a sandy bed from the current, on account of its density and thickness, and having, withal, considerable inherent strength, caused it to be at once used at other river points where, in some cases, other types of mattress had failed in the first year's trial.

In 1885, the continuous woven mattress was adopted by Major (now Colonel) Charles R. Suter, Corps of Engineers, U. S. Army, President of the Missouri River Commission, M. Am. Soc. C. E., as part of the standard Missouri River revetment, and continued to be used as such on all revetment work constructed under the Commission until that organization was abolished by law, on June 13th, 1902. It is still used for local improvements on the river above Sioux City, Iowa. It also became the standard type of footmattress for protecting permeable dikes against scour, and was

FIG. 1.—COARSE WOVEN MATTRESS DIKE SCREEN.

FIG. 2.—LONGITUDINAL WITHES IN SELVAGE OF WOVEN MATTRESS.

used to some extent for screen work, supported on pile frames, for closing chutes. Fig. 1, Plate XXXIII, exhibits a coarse woven mattress used for the latter purpose. It was 50 ft. wide, and was successfully woven and sunk in a very swift current. Its purpose was to make possible the construction of a dam, about 200 ft. below the screen, in water about 25 ft. deep. Mr. Yonge.

The plan of protecting the bank above low water with woven mattress on which stone was laid was not entirely abandoned until about 1887.

Green willow withes are most suitable for mattress weaving, although dogwood and saplings of other varieties can be made to serve that purpose. In weaving upper-bank mattress, the withes are stuck into the bank slope.

The following is a description of the process of weaving subaqueous mattress. The illustrations, Figs. 2, 3, 4 and 5, Plate XXXIV, are from photographs of rude models, for conveying an idea of the steps of the process, but not of the product.

The weaving can be started either by sticking withes into the toe of the bank and weaving out over the mattress-boat ways, a selvage, described below, being formed across the head of the mattress; or by laying across and lashing temporarily to the ways a long fascine of 8 or 9 in. diameter, to serve as a header and hold the first two or three series of withes. The latter plan, the one usually followed, is incorporated in the description.

Through the fascine, the first series of pairs of diagonal withes are thrust so as to stand at an angle of elevation of about 40° and have their included angles at the fascine about 70 degrees. The diagonals are then worked over and under each other at their alternate intersections, forming a loose, rough network of diamond-shaped meshes about 14 by 10 in. (see Fig. 2, Plate XXXIV). A second series of pairs of withes is then thrust through the fascine, approximately parallel to those of the first series, and bisecting the first line of meshes. If the withes are small, or a close mattress—generally a desideratum—is to be made, a third series of withes is stuck through the first line of meshes into the fascine, though not so shown on Fig. 4, Plate XXXIV, reducing still further the spaces of the initial meshes. The fourth series of diagonals is then thrust through the second line of original meshes, so as to reach under the fascine. Next follows the fifth series, the members of which are thrust through the third line of original meshes, or about 14 in. behind the fourth series of diagonals, so as to extend about 2 or 3 ft. under the completed weaving. The weaving is proceeded with by sticking each succeeding series of diagonals about a mesh's length back of the preceding series.

If it is desired to thicken the mattress, the distances between the

Mr. Yonge. lines of the series of diagonals are reduced, and contrariwise. The angles with which the weaving was started are approximately observed throughout the process.

Simultaneously with the placing of the second series of diagonals, the first stitch or loop of the selvage is formed by sticking one or more small withes, herein referred to as selvage withes, through the outer corner mesh of the mattress and through the fascine. These withes are then bent around and lashed to one or more diagonals of the first series. The butt ends of withes for the second loop are passed under the first loop and through the fascine, and the brush ends are bent down and around and lashed to a diagonal. Each succeeding loop is formed by sticking the butt ends of the selvage withes along the outer side of the selvage—6 to 10 in. apart—first under the preceding loop, then over the loop back of the preceding one, and finally under the completed weaving, the tops being bent over, turned in and lashed to the diagonals, as already described. The selvage is thus made up of a series of intersecting scallops, each of which crosses the preceding one at about its middle, the butt ends of the withes being held by the preceding scallops and the diagonals in the body of the mattress, and the brush ends by lashings and the weaving in contact with them above and below.

Fig. 5, Plate XXXIV, is intended to show the weaving of the selvage. The weaving thereon shown for the body of the mattress does not accord strictly with the method described.

As the weaving of the selvage proceeds, the parts of most of the diagonals protruding beyond it are made to form a part of it by turning them in with the selvage withes. If the weaving of the body of the mat is close, the selvage withes are sometimes omitted, the protruding ends of the diagonals taking their place. The selvage is also generally thickened and reinforced by weaving withes into it longitudinally, parts of the longitudinals being covered by the turned-in ends of the diagonals and selvage withes.

After several lines of meshes in the body of the mattress are woven, the withes are held down by the weavers sitting on their free ends. Care must be exercised to have the parts of the mattress made by contiguous weavers properly united.

Although the general appearance of the completed mattress is symmetrical, the meshes are not of uniform size. This, however, is not essential. It results, to a great extent, from the withes being of different sizes.

An important requirement is that of thrusting each withe 2½ ft. or more under the completed weaving. In this type of mattress each withe is under strain, binding the preceding ones and being bound by those that follow. It is next to impossible, without breaking it, to pull a withe from a completed mattress.

PLATE XXXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 995.
YONGE ON
MISSOURI RIVER IMPROVEMENTS.

FIG. 1.—PANORAMIC VIEW OF MISSOURI RIVER AND PERMEABLE DIKES.

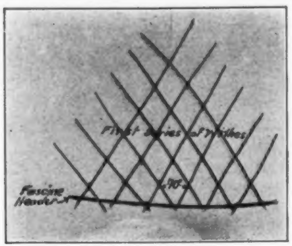


FIG. 2.

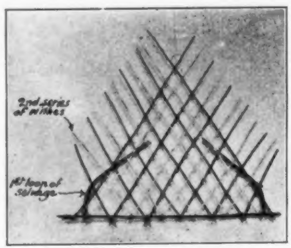


FIG. 3.

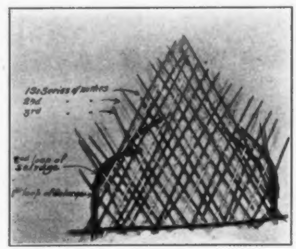


FIG. 4.

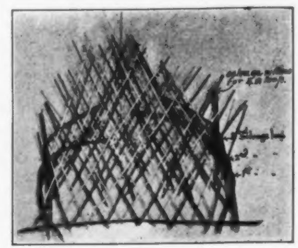


FIG. 5.

MATTRESS WEAVING.

Before sinking the mattress the fascine header should be well lashed to the adjacent weaving with galvanized-iron wire. Mr. Yonge.

This type of mattress is used nowhere except on the Missouri. It differs radically from the types used on the Mississippi, in which the basis of strength consists of wire net, cable, or iron rods. In the woven mattress the strength is derived, in great measure, from the resilience of the bent withes.

A few weeks' practice will make proficient weavers. An average 8-hour day's work for a weaver is about 800 sq. ft., for which from 5 to 5⅝ cords of willow brush, delivered in bundles, are required for mattress about 12 in. thick.

Fig. 2, Plate XXXIII, which is a view of part of a dike-mattress afloat, furnishes an example of longitudinal withes in the selvage. Fig. 1, Plate XXXIII, shows the selvage without this reinforcement.

In 1887 the writer experimented with an upper bank protection of stone rip-rap without mattress, covered with quarry spalls.* This detail was adopted for the later revetments constructed under the Commission. The writer, however, used spalls or macadam both under and over the stone paving. He prefers the former to the latter order, as the smaller stone is thereby prevented from being swept away by moving ice or swift currents, and in that position also thoroughly shields sandy bank slopes from the "suck" of the waves. The writer, about three years ago, successfully adopted a somewhat similar plan in the construction of an inclined sea-wall, formed of concrete blocks with open joints, laid on a backing of broken stone, covering an earth embankment.

The last paragraph under the caption "Foot-mattress," on page 301 of the paper, refers to the placing of anchor piles (to receive wire strand stays for mooring the foot-mattress while being woven and partly ballasted). The stays referred to are marked *b* on Fig. 6, and are referred to below as surface stays. They were attached at one end to the anchor piles, *a*, at 2 ft. or more above the water, the other ends being made fast along the up-stream edge of the mattress at intervals of 10 to 15 ft. The office of the stays was to prevent the part of the mattress between its up-stream edge and the dike piles from bagging, especially while being ballasted, and then being folded by the current and carried against the dike piles. Experience soon showed that where the current was very swift the surface stays alone were not always sufficient to insure the sinking of the mattress, so as to lie even on the bottom. At some of the dikes on the Osage Division of the first reach, therefore, where the conditions for mattress sinking appeared to be unfavorable, additional stays, marked *c* on Fig. 6, and referred to as ground stays, were used. These were attached to the anchor piles by a bight, made at their ends, which was forced down the pile to the ground.

*Annual Report, Chief of Engineers, U. S. Army, 1888, Part 4, p. 2365.

Mr. Yonge. On the completion of the weaving and the placing on the mattress of sufficient stone to start its sinking, the surface stays were cast off. The ground stays, meanwhile, prevented the mattress from being rolled and torn by the current while sinking to the bottom, after which the ballasting was completed.

The double system of stays was also used in sinking revetment mattress when the weaving of a section was started in a swift cur-

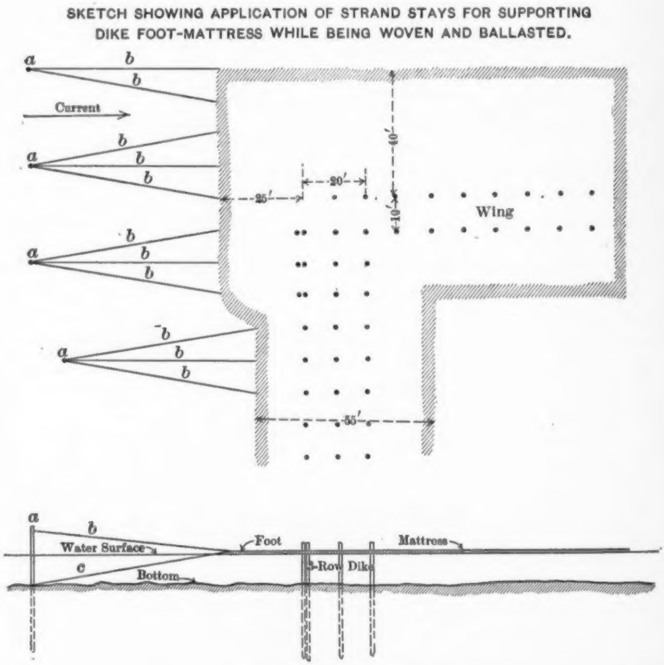


FIG. 6.

rent. The anchor piles were given moderate penetrations, and were generally scoured out. Those in the navigable channel were broken off below the ground by the use of a steamboat.

The objection to using solid spur dikes for the improvement of the Missouri and other rivers of its class, referred to in the beginning of this discussion, arises from the effects of the head resulting from stream impact. The initial effect is a rapid flow around the

stream end of the spur, varying in intensity and extent with the head, and producing vortical action above and below the spur. Mr. Yonge.

In consequence of the swift current around the spur, heavy scour occurs, resulting in the formation of a gorge section, toward which the flow lines converge, the flow finally becoming a contracted vein. Meanwhile, the adjacent banks, above and below, are attacked by vortical flow and are undermined.

The scour in front of the spur, and also on its sides, causes the mass of stone to settle indefinitely, the downward movement being aided by the weight of the spur on the saturated sandy bottom. This subsidence is not checked until a material is reached which will resist scour, usually bed-rock. The adjoining banks recede, because of undermining, until they attain a condition of comparative stability, the extent of the recession depending principally on the distances between consecutive spurs, their projection from the bank, and the angle or direction of flow.

While the above-described forces operate, the spurs must be kept built up and extended shoreward by greater or less expenditures of stone, if they are to be preserved and their severance from the bank prevented.

The final result, if the maintenance of a series of spurs is persisted in, is the creation of a deep, tortuous, narrow channel, with a high velocity, beset with cross-currents and eddies, in navigating which, except possibly at low stages of water, the skill of the pilot is put to the test to save his boat from disaster.

Assuming that the distances between spurs are diminished, the above objectionable conditions are correspondingly modified, and they finally disappear when the bank protection becomes continuous, maintaining a channel of approximately uniform depth and velocity. The only valid objection to a stone revetment, maintained until stable, is its great cost. During the three years ending with 1870 the river bank in the bend between the Hannibal and St. Joseph Railroad Bridge and the mouth of the Kansas River was protected by a revetment of rip-rap, to make permanent the channel approach to the bridge draw span. The cost of the work, including its maintenance, is estimated by Octave Chanute, Past-President, Am. Soc. C. E., who was the Chief Engineer of the bridge, not to have exceeded \$100 000. The length of bank protection is also estimated by Mr. Chanute at about 8 300 ft.

There will be great variation of cost of stable stone revetments, depending principally on local physical conditions.

For a free sandy bank greatly exposed to the current, impinging even at a small angle, with bed-rock at a depth of about 60 ft. below low water, the expenditure of stone required to bring the work to a condition of stability would probably amount to 50 cu. yd. per lin. ft. of bank.

Mr. Yonge. The protecting influence of a solid spur does not generally extend down stream more than three or four times its normal projection from the bank, and to this extent only when the projection is sufficient to keep the area of vortical action at some distance from the bank. The protection afforded by spurs of slight projection, as has been pointed out, is to a great extent negative.

If spurs are flanked by the stream, they obstruct the channel and thus hasten the erosion of the bank they were designed to protect, and finally either disappear in the sands of the river, or, if their subsidence ceased before they were detached from the bank, they in due time form part of the extension of the opposite shore. Several examples of solid spurs, constructed by corporations, being flanked by the river came under the writer's observation while the improvement of the Missouri by the General Government was in progress, a description of one of which follows.

In 1898 the writer found and identified on the convex side of Belmont Bend one of three spur-dikes placed on the concave side of the bend in or about 1872, to arrest the erosion which it was apprehended would terminate in the formation of a cut-off through the peninsula formed by the windings of the river, and thus render valueless the recently constructed St. Joseph and Grand Island Railway Bridge, which joined the peninsula to the opposite shore below the bend. In 1881 the spurs were flanked by the river, and the bank to which they had been attached receded about 1 500 ft., until held, in 1891, by Government revetments.

As the protective zones of short revetments and solid spurs are very restricted, they cannot be used successfully in a general scheme for improving a river with sandy banks, if bank protection or channel regulation are the ends to be attained. The most effective protection for the former purpose is a stable continuous revetment; for the latter, either a continuous revetment or a properly designed system of permeable dikes, made permanent by proper protection at their stream ends, in which the objectionable vortical action incidental to solid spurs is greatly lessened by the flow through the screen work.

The foregoing remarks as to the use of solid spurs are intended to apply solely to rivers with sandy, or other very friable banks.

Permeable dikes with curved stream ends are more costly than straight dikes, and have the serious disadvantage of inducing a rapid flow along their upper face, which prevents the formation of battures above them. Those constructed did not prove to be superior to straight dikes in resisting the forces to which dikes of both forms were subjected.

On page 298 Mr. Fox states:

"The practice, as to height of dikes, differed on the two divisions, and varied from time to time; but, in the main, they were run out

level with the top of the bank, or 2 ft. above standard high water, to near the standard high-water contour of the proposed rectified shore, and from there they sloped down to as near 2 ft. above standard low water as the stage of river at the time permitted." **Mr. Yonge.**

In designing the dikes for the upper division (Osage) of the first reach, their inshore ends were fixed at, or slightly below, standard high water, or at the same elevation as the bank, if it was below the above reference plane. The main banks on the Osage Division are generally slightly above standard high water. Variation of dike profile was tried, to discover, if possible, whether one form possessed any advantage over another, a dike's crest sometimes being given a uniform inclination throughout, or some parts were made horizontal and others inclined. A rule almost invariably observed, however, in planning the dikes, was to place their crests so that the area above them to standard high water, with the area below the same plane in the proposed rectified channel, *i. e.,* between a pair of opposite dikes, or between a single dike and the opposite shore, together with the area which could reasonably be expected to result from scour, would be adequate for passing, at a normal velocity, the discharge for a stage of standard high water. The range between standard low water and standard high water was determined to be 13.8 ft., and the low-water width and channel areas required for discharge at the above stages, obtained by observation of natural, approximately stable reaches, were determined as about 1 000 ft., 9 000 sq. ft., and 40 000 sq. ft., respectively.

For a straight reach with an ideal parabolic section, the maximum scour at mid-section entailed by the above estimate would have been 15 ft., and the total center depth at the higher stage 42.3 ft.

Examples of the river's channel enlargement under contraction were not wanting to show that the estimated increase of area required in the projected rectified channel, estimated at about 10 000 sq. ft., would be realized. A part of this enlargement, varying within wide limits, is acquired, in the unimproved parts of the river, during every flood and lost by fill-back as the stage declines. On some parts of the Osage Division the old channel was closed and an entirely new channel formed, the river being first trained to initiate the excavation of the new channel through the medium of the dikes, and then forced, by the use of additional similar dikes, to enlarge it. As the dikes were permeable, the extent to which at first they reduced the channel section was considerably less than assumed in the calculation, *viz.,* the polygonal areas of their side elevations. The contracting effect for causing channel scour also increased gradually as the battures formed, and built up by deposition, the battures in due time becoming the real factors in perpetuating the regulation begun by the dikes.

Mr. Yonge.

During the first two or three years of the work the proposed enlargement of section progressed satisfactorily, and the indications were favorable for the attainment of the desired enlargement as soon as the battures should be fully formed. There are no published data showing the extent to which the high-water section on the Osage Division was finally developed.

To avoid a too sudden channel contraction, the dikes were not always at once constructed for their proposed full length. An interesting illustration of this feature is exhibited by Fig. 1, Plate XXXIV, showing a group of dikes designed to rectify a part of what was known proverbially as the worst reach of the river. The total projection from the (right) bank of the dike in the foreground is about 1 700 ft. It was constructed in five sections, one or more floods occurring in the intervals between the extensions. The period covering the dike's construction was about 27 months; the actual time of its construction was less than 2 months.

It will be noticed that the continuity of the pile structure is broken by four offsets separating the several extensions. The offsets were made to avoid the pot-hole scoured at the end of each section after its construction, the crossing of which would have required very long piles and made the exposure of the piles above ground too great to resist overturning by the pressure of the current transmitted through the accumulation of drift-wood above the dike.

It will be of interest to mention that this dike, in conjunction with several others above it and on the opposite bank, effected a very remarkable change of position of the main channel, which, before the improvement was begun, followed the right bank to the vicinity of the site of the dike, where it divided, one part flowing to the right around the point to the Osage River, the other crossing a middle ground or reef to a secondary channel following the left bank. The latter was the channel generally used by steamboats, and the depth on the middle reef was sometimes only 30 in.

The width of the river between banks is about 3 600 ft. The view shows the improvement in a state of forwardness, the flow being concentrated in a channel about 1 050 ft. wide, with a least depth of 6 ft., although the battures, especially those adjacent to new dike work, were not fully formed.

Mr. Chittenden.

H. M. Chittenden, M. Am. Soc. C. E. (by letter).—This paper is of particular interest, not only because of its merit as a contribution to the science of the improvement of navigable waterways, but because it marks the close of an important chapter in the history of inland navigation in the United States. The Missouri River, both in its physical characteristics and in its commercial history, is one of the remarkable streams of the world. It is a large stream absolutely, though a small one considering the

Mr. Chittenden.

area of its water-shed. The extreme flood discharge of the Ohio River at its mouth is probably about 5 cu. ft. per sec. per sq. mile of water-shed, while that of the Missouri barely reaches 1.5 cu. ft. At Sioux City, where the river may be considered as emerging from the arid and semi-arid region into the humid region, the extreme low-water flow in the dead of winter, when the tributaries are nearly all ice-bound, is about 7 000 cu. ft. per sec.* The maximum discharge at the same point is that of 1881, which is estimated at more than 600 000 cu. ft. per sec. At the mouth of the river the discharge probably falls as low as 15 000 cu. ft. per sec. in winter, and is believed to have reached approximately 900 000 cu. ft. per sec. in the great floods of 1844 and 1903.

The length of the river from its extreme source at the head of Jefferson Fork to its junction with the Mississippi is 2 945 miles; and to the mouth of the latter stream 4 221 miles—the longest river in the world. More remarkable than its actual length, however, is the great length of its navigable channel, which is 2 316 miles from the extreme point reached by steamboats near the Great Falls of the Missouri to the mouth of the river, and 3 592 miles to the Gulf of Mexico. Perhaps most interesting of all is the fact that the river in its natural condition, without any artificial improvement whatever, was navigable for steamboats over all this distance. During the entire period of its greatest use as a navigable waterway the Government did almost no improvement work upon it. The river was, in fact, practically an inclined plane, without sensible interruption by cascade or rapid, rising on a gentle but gradually increasing slope (less than 3 in. per mile at high water near the Gulf to more than 3 ft. near the Great Falls) until it attained an altitude at the head of navigation of more than one-half mile above the level of the sea.

Still another remarkable fact, to those unacquainted with the history of this river, is the extent to which it has been utilized in commercial navigation. The period of its use for this purpose is now nearly, if not quite, 200 years, and it has been navigated by steamboats for 85 years. From 1820 to 1880, it was a potent factor in the development of the trans-Mississippi territory, and for about 25 years, from 1845 to 1870, its importance in this respect can hardly be over-stated. It was the one great highway to all the western country. Were there to-day but a single railroad from the Mississippi River to the Rocky Mountains, its commercial value to the tributary country would be relatively no greater than that of the Missouri River during all this period. All operations in the far western country were planned with reference to it, and it was a controlling factor in the great movements which led to the sub-

* As measured under the direction of the writer on February 16th, 1905, after a protracted cold spell of nearly two months over-spreading the greater portion of the water-shed.

Mr. Chittenden. duing and final settlement of the West. The war with Mexico, the exodus of the Mormons, the vast migration to California, the later rush to Montana, the military subjugation of the Plains tribes, and the work of occupying and settling the vast Empire of the West, all relied to a great extent upon the aid of this mighty stream. Statistics have never been preserved showing the actual magnitude of this business, but it seems certain, from such information as is available, that it must have exceeded that of either the Ohio or Upper Mississippi, and probably of both combined.

As already stated, the greater part, in fact nearly all, of this extensive commerce was carried on without material assistance from the Government in improving the navigable channel. Except in the matter of limited expenditure in the removal of snags along the lower course of the river, the Government did no channel work until the period of active boating had passed. As in the case of other tideless rivers of the United States, but to a more pronounced degree, the advent of the railroad proved the ruin of the river business. The difficulties of navigation on the Missouri have always been very great. The rapid current, the excessively shifting channel, and the prodigious accumulation of snags made boating at all times hazardous and uncertain. The number of steamboat wrecks in the history of the river is close to three hundred. High freight rates were necessary to insure against these dangers. The railroads cut these rates, and, by their greater convenience and certainty of operation, ruined the boat business. It was practically gone by 1880, about the time when the Government turned its attention seriously to the river.

It will naturally be asked why the tendency of events was not more clearly recognized at the time, and with it the inadvisability of spending great sums upon a work the utility of which was so much open to doubt. The answer is found in the extreme reluctance of the people along the valley to admit that the transportation route which had served their country longer than they could remember had at last ceased to be of value. They could not become reconciled to the fact that the steamboat must go. They longed for a revival of the business, and as it could not be under the conditions then existing, they asked for such an improvement of the channel as would enable the boats to compete with the railroads. They resolutely shut their eyes to the logic of events, and never fully realized the magnitude of the obstacles that stood in their way. They failed to appreciate the enormous physical difficulties of regulating the Missouri River so that it would maintain a navigable depth by scour. But, even more important, they failed to grasp the broader commercial features of the problem, *viz.*, that the river was a less convenient line for shippers; that it was wholly useless for passenger

Mr. Chittenden.

traffic where railroads were available; that it was entirely closed for several months of every winter; that over much of its course it did not flow in the right direction nor reach the most important centers of activity; and that, therefore, it could not adequately serve the ever increasing demands for transportation. All these considerations were ignored or overlooked, and it was decided to attempt to restore the river to its former high standing as a commercial highway. In the effort to realize this purpose, a vast expenditure of public money was made and a prodigious amount of work accomplished. The technical details are given by Mr. Fox in this paper. But the outcome was not what was desired. The waning commerce of the river continued to dwindle, without any reference to the heroic efforts being made to keep it alive, and, finally, the Government gave it up as a hopeless task.

Another reason which led to its abandonment more quickly than would otherwise have been the case was the general policy under which the work was conducted. To carry out the purpose of Congress a commission was created representing the interests of the valley. Adhering strictly to the theory of river and harbor improvement, *viz.*, that public expenditures on such works are authorized solely in aid of interstate commerce, the Commission logically undertook to carry on its work systematically and continuously from the mouth up stream, and always disapproved the diversion of any portion of its funds to individual localities not yet within the field of systematic work. This was the natural, logical method of attacking the physical problem. But it failed to grasp the practical aspects of a public work in which the people of the whole valley were interested. When the communities remote from the field of actual work came to understand that the systematic improvement might not reach their neighborhood for a generation to come, they became restive and dissatisfied, and succeeded in inducing Congress to divert so large a portion of the appropriations to special localities as to cripple effectually the Commission's scheme of continuous improvement.

A third condition that operated to the same end was the failure to recognize the urgent necessity for a certain class of work which might have been carried on advantageously with the channel improvement. This was the protection of the river banks from erosion. The Missouri River is undoubtedly the most destructive stream, in the matter of bank erosion, that exists on the globe. This is a sweeping statement, but the writer believes it to be correct. The evil is of a character which is extremely difficult to circumvent by individual effort because of the fact that the agencies at work are so powerful and reach so far that the individual land owner, unless a great railroad system, is absolutely at their mercy. It has always

Mr. Chittenden. seemed to the writer that the Government, in its improvement work, could very properly co-operate with the land owners, and, wherever not incompatible with the maintenance of the navigable channel, could plan its work so as to protect the immensely rich and fertile bottom lands from the ravages of the river. The writer will presently give his reasons for thinking that the works best adapted to the maintenance of a good depth of water are those which would best serve the purpose of bank protection. With such a policy established, private parties would contribute materially to the protection of their property, and the expenditure of public money would yield a vastly increased benefit over what it would if confined exclusively to channel improvement.

In view of the recognized limitations upon river and harbor appropriations, the Commission, of course, was entirely consistent in refusing to recognize bank protection as any part of its plan, except as it related exclusively to the improvement of the navigable channel. Local interests, on the other hand, felt that it was only by special allotments that they could derive any benefit from the appropriations made by Congress. The situation thus developed could not continue, for it was in direct opposition to the official plan of the work, and involved too great a departure from the theory of river and harbor legislation; for, while these special appropriations were made for the "improvement of the river" at the localities designated, this term was virtually a euphemism to cover up the real purpose in the protection of riparian property. As before stated, it has seemed unfortunate to the writer that the existing conditions could not have been recognized on their merits, and a system of improvement have been inaugurated, which, in co-operation with the efforts of land owners, would have been of inestimable value to the communities along the river. But since it could not be, or at least was not, so recognized, it was clearly the duty of Congress to withdraw from the river altogether, for the demands of navigation were not of themselves sufficient to justify any further large expenditure. The abandonment of the improvement practically took effect in 1902 when the Missouri River Commission was abolished, and it has fallen to the writer, in his official capacity, to superintend the obsequies of this once very considerable public work.

It should not be concluded that, because the policy of improving the Missouri has practically been abandoned, the expenditure of so much public money has been in vain. The paper of Mr. Fox indicates something of what it has bequeathed to the people of this valley and to those of the valleys of other similar streams. Probably few problems in river improvement have received the persistent, intelligent and painstaking study that this has. The characteristics of the stream were exhaustively studied. As the magnitude and cost

Mr. Chittenden.

of the problem began to develop, every effort was made to devise more economical and efficient methods of controlling the river; and it is doubtful if future experience will evolve anything of value, so far as the control of this river is concerned, that has not already been tried in these investigations. A large share of this work has been done by Mr. Fox himself, and the science of river engineering has been distinctly promoted by his efforts.

Among the valuable results of the Government work on the Missouri River may first be noted the surveys and maps of the valley. These cover the entire length of the stream from the Three Forks to the mouth. They are based upon a system of secondary triangulation and a line of precise levels. The surveys have been published in a set of eighty-five sheets with nine index sheets and two containing titles and explanatory data. The scale is one mile to the inch. There are also unpublished large-scale maps extending from the mouth to above Kansas City, and a great number of maps of special localities. Altogether, the data of this character available for public use are of great value. They are constantly called for, and are serving a most important purpose.

The various types of works developed for controlling the river will find wide application upon all alluvial streams, and especially upon the Missouri itself; for the dwellers of the valley will always have with them the problem of self-preservation from the ravages of this stream, and they will find their greatest aid in the fund of information which the Government has provided for them through years of costly experimentation.

In criticism of the works themselves and the several types of construction made use of, the writer has drawn this conclusion, though somewhat at variance with the practice of the Commission: That those works which are built out boldly into the stream, at right angles, or nearly so, to its current, are less effective in improving the channel and in protecting property than those built approximately parallel to the current. The theory of the first class of works is, of course, that they contract the channel, force a larger flow through a smaller width, and thus compel the current to scour out a greater depth.

Two conditions of great moment operate against this system: First, there is the destructive energy of the river in flood with its enormous volume of water, its rapid current and its load of ice or drift. Ordinary dike construction, sooner or later, and generally sooner, yields to these agencies. The scour around the ends weakens their resisting power and causes them to succumb more rapidly. Bank-heads are less exposed to ice and drift, but are equally open to attack by scour. In the second place, this method of control seems to the writer to possess grave defects in principle. The natural

Mr. Chittenden. effect of such non-yielding structures thrust bodily into a stream is to disturb and demoralize the flow. In an alluvial bottom, which yields almost to the touch of flowing water, these disturbances are likely to have far-reaching effects, throwing the river out of the alignment intended, and producing results quite different from those desired. Mr. Fox has referred to some of these defects on pages 305-307 of his paper.

In contrast with this method of improvement by force, is that of longitudinal works, such as bank revetment or dikes parallel with the current. These avoid the drawbacks mentioned above. The ice and drift rarely lodge heavily upon them. They do not disturb the flow of the river, but by their smooth and even alignment seem actually to attract the flow in their direction and to maintain in that way an increased depth and uniformity of section. They accomplish their purpose of improving the channel by coaxing rather than by driving.

From the nature of the two classes of work, revetment is more permanent than dike work. The latter being largely of wood, partly beneath the water and partly above, must inevitably fall by natural decay in a comparatively short time. Revetment work has no perishable material above water, and, therefore, is largely exempt from the processes of decay. Dike work is exposed to the direct onslaught of the river, with its ice and drift and rapid current. The revetment rarely, if ever, receives the attack directly, but at such an oblique angle that it glances off with comparatively little impression. Dike works are avowedly for the purpose of changing the flow of the river; their influence is far reaching; and the current may be thrown against other banks, causing new destruction and giving rise to just complaint. The revetment never has this effect, but, on the other hand, tends to hold the river in its existing channel.

An exception to the above criticism upon spur-dikes is found in the case of short dikes of, say, 100 ft. in length, placed comparatively close together and built on a steep grade from the top of the bank to about low water at the outer end. These dikes serve admirably the purpose of bank protection, are but little exposed to ice and drift, and, by their closeness to each other, do not disturb the flow of the river as much as do the long, high dikes spaced at greater intervals. Their cost, moreover, is only about half that of standard revetment.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is greatly impressed by this very instructive paper. He is especially interested to learn the striking likeness that the Lower Missouri River bears to the abnormal physical features of the Feather River and Lower Sacramento River, in California.

Mr. Le Conte.

In 1876 the writer was engaged in building temporary wing-dams on the Feather River, between Marysville and the mouth, with the view of ameliorating navigation for the season as much as possible. The physical conditions of the stream, overloaded with mining débris, certainly beggars all description. The experience gained while working on this river was so remarkable and novel that he observed conditions carefully and took full notes of all the physical features peculiar to that river. It did not take the writer very long to arrive at the conclusion that the great over-mastering physical feature which dominated everything connected with the improvement of the channel way was undoubtedly the progressive bed-flow of sand. This feature wiped out, so to speak, the ordinary hydraulic characteristics entirely, and introduced an altogether new controlling element which was extremely difficult to deal with, chiefly on account of the utter lack of knowledge regarding it.

Generally speaking, it may be said that in the bends the water surface was not more than 90 or 100 ft. wide and from 6 to 10 ft. deep, and the flow was quiet and smooth. On the crossings, however, the flow was fully 600 ft. wide with a uniform depth of only from 12 to 14 in. Such a river section as this is truly phenomenal. The entire surface was so-called "spotted water," giving indisputable evidence of a general bed-flow for the entire width of 600 ft. In wading over this crossing, in rubber boots, one has to keep moving all the time, as standing in one place too long would probably lead to his gradually sinking down out of sight. The sandy bottom was all alive and moving down stream. The temporary wing-dams were generally built on these crossings to concentrate the water flow, and 24 hours after construction a dry sand bar 1 000 ft. long by 400 ft. wide would form below the dam, the fill above the dam being much less in size. These dams were at first built 6 ft. in height throughout their entire length, but they gradually settled down into the moving sand until at the outer end they practically reached the water surface; at the same time, the entire dam was swinging down stream radially some 20° from the alignment first built upon, and then the dam apparently settled down and came to a stand. This would seem to indicate, roughly, that only the upper 6 ft. of the bed-flow was moving at low stages of the river.

Unfortunately, circumstances prevented the writer from observing the Feather River during flood stages, but he did manage to observe the flood stages on the Lower Sacramento River near the head of Steamboat Slough. The facts observed there are very interesting and highly instructive. When the river was bank full, and flowing with a surface velocity of 7 ft. per sec., soundings were taken across the channel, and the results when plotted showed that the sandy bottom had come up just about as much as the water had

Mr. Le Conte. raised on the gauge; in other words, the high-water sectional area was only a very little more than the low-water sectional area. But the real difficulty was in obtaining good reliable soundings during high-water stage, the sandy bottom feeling like a pot of boiling mush, so that in any cast of the lead one could not tell within 3 ft. of what the sounding ought to be; indeed, the writer has really very serious doubts as to whether the high-water sectional area, in point of fact, was any greater than the low-water sectional area. This condition continued during the entire time of his observation, and, therefore, was not a mere transition stage of the river, or due to the passage of a sand wave. In such rivers as these, of course, the usual method of gauging the river discharge, by the gauge height readings, is out of the question entirely. He was much pleased to note the magnificent results obtained by the Missouri River Commission at the forty-five mile stretch near Jefferson City. This work is a masterpiece of good river engineering, and will always stand as a monument to the ability and skill of the Engineer Officers in charge.

The excellent details supplied by the author are full of interest to the river engineer, who instinctively depends upon actual experience as the best guide and most reliable teacher. In fact, it may be said that his practice is all based on experience gained by hard work and costly experiments. What is good for one river may or may not be suitable for another; so much depends upon local conditions and the materials that are available and handy. The remarkable feature brought out clearly by the author, namely, the universally divided flow in the improved channel, is a most troublesome fact which adds another item to the long list of difficulties to be encountered. When a broad, general and persistent fact like this is met there is always a simple cause for it. Although the writer is not prepared to speak positively, yet, judging from his past experience, he is inclined to think that this feature is a natural phenomonen of bed-flow, and that the mid-channel shoal is the characteristic feature of this bed-flow.

It is humiliating to note how little the public appreciates the true intrinsic value of improvements to river navigation. The river, after all, is the only true friend and freight controller the public has. The public should never be so blind to their own interests as to overlook systematic river improvements.

Moreover, the wealth added to the adjoining country, entirely incidental to the improvement of navigation, is a matter which should not be overlooked. The writer is credibly informed that in a distance of 18 miles, near Jefferson City, new land formed by improvements equals 5 500 acres, and the area of land protected by the same improvements equals 12 800 acres. This would fairly represent the sum of $915 000, or more than $50 000 per mile of river. This amount, in itself, would practically pay for the improvements.

S. Waters Fox, M. Am. Soc. C. E. (by letter).—The discussion of this paper, by Messrs. Yonge, Chittenden and Le Conte, has added much of interest and value to it. Mr. Fox.

The elaboration, by Mr. Yonge, of some of the details of construction, and the remarks of the other two gentlemen, particularly those of Mr. Chittenden relating to the economic aspect of the improvement of the Missouri River by the General Government are of especial interest.

In reply to Mr. Yonge's remarks about the comparative merits of the different forms of upper-bank protection work, the writer's opinion, after careful observation of the results of experiment with a great variety of forms, is that the standard specifications, as given in the paper, if faithfully carried out in detail, will fulfill all requirements against wave "suck" or wash, dislodgment by running ice or other abrading force, and gulleying by overpour from surface drainage.

Mr. Chittenden's analysis of the causes which led up to the abandonment by Congress of the work of systematic improvement of the Missouri River is strikingly clear and correct. The writer cannot too strongly endorse his views as to the propriety of the adoption by the General Government of a policy of protecting the banks of the river. There are few measures which Congress has been called upon to provide funds for that offer such large and sure returns. As a measure for the improvement of the navigable depth of the river, it would be successful. As a business proposition, it would pay in the enhanced value of the lands and property directly protected, to say nothing of the value of the land that would thereby be reclaimed from the river.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 996.

THE COMPENSATING WORKS OF THE LAKE SUPERIOR POWER COMPANY.*

By G. F. Stickney, Assoc. M. Am. Soc. C. E.

With Discussion by

Messrs. L. J. Le Conte and G. F. Stickney.

Lake Superior, the most westerly and the largest of the Great Lakes, covering an area of about 32 000 sq. miles, has a mean elevation of 602.5 ft. above sea level. The outlet of the lake is through Lake Huron, by way of the St. Mary's River, with the Province of Ontario on the north, and the State of Michigan on the south. At the head of the river is a rocky barrier, forming the "Soo" Rapids, which have a fall of 19 ft. in a distance of 3 600 ft. At the upper end of the rapids the river is 2 400 ft. wide, and is crossed by a railroad bridge of ten spans. This bridge connects the two cities of Sault Ste. Marie. The outflow through St. Mary's River, according to the best information available, varies from 56 157 to 125 147 cu. ft. per sec., with an average flow of about 92 000 cu. ft. per sec., the volume depending upon the stage of the lake.

The location is unusually favorable for the development of water-power, and advantage of the situation has been taken by the Lake

* Presented at the meeting of March 15th, 1905.

Superior Power Company to establish various industrial plants on the Canadian side, where a water-power canal was already in operation. On the American side there was a small electric light plant, utilizing water-power. The Lake Superior Power Company was preparing to develop 40 000 h-p. on the American side, and, for this purpose, had under construction a canal and a power-house.

The canal commences above the rapids, and is excavated through the City of Sault Ste. Marie, Mich., returning to the river below the rapids, with a total length of 2 miles. It is 200 ft. wide, 22 ft. deep, and the water will have a velocity of 7 ft. per sec., furnishing 32 000 cu. ft. per sec. at the wheel pits, with an effective head of 16 ft. Thus, the quantity of water to be withdrawn from the lake is about 35% of the mean average flow over the rapids. The elevation of the surface of Lake Superior varies within a limit of 2.4 ft., depending upon the rainfall and the season of the year. If an artificial outlet were provided, increasing the natural discharge 35%, it is evident that the lake would be lowered, gradually, until such time that the depth in the channels was reduced to a stage that would just accommodate the flow of surplus water, and the lake would remain at this low level. Such a change, in addition to reducing the power output, would decrease the depth in all the channels and harbors of Lake Superior. To provide against such a contingency, regulating works, at the head of the rapids, were necessary, to restrict and to control the flow of water in the natural channel and to compensate for the volume taken out through the water-power canal. A wing-dam, or some similar structure, closing up part of the natural channel, would effect this compensation, if the channel closed were equal in area of cross-section to the water-power canal, but such a structure, adjusted to the mean flow, would increase the low-water discharge and would decrease the high-water discharge, thus extending the range of fluctuation of the lake level. A more flexible method of compensation was necessary, to avoid antagonizing the vast interests on the lake, which would certainly oppose any change of the natural conditions. It was required, therefore, to design a structure which could be adjusted to present a greater or a smaller obstruction in the channel, according to the stage of the lake, which would compensate at all times.

Under the direction of Alfred Noble, Past-President, Am. Soc.

C. E., Consulting Engineer, plans were prepared for regulating works, including four sections, as follows:

1.—An embankment,
2.—A movable dam,
3.—A crib dam,
4.—A submerged weir.

Each section was to be about 242 ft. long, corresponding to the length of the spans of the International Railroad Bridge, and the works were located on a line parallel to, and 150 ft. above, the center line of that structure, opposite the first, second, third and fourth spans, numbering from the Canadian side.

The embankment is a simple fill across a shallow part of the river, 12 ft. wide on top, with side slopes of 1½ to 1, made of earth and stone, with height sufficient to prevent its being overtopped by high water during a storm.

The movable dam, Fig. 1, which is the chief feature of the design, is intended to furnish the flexible element. Several different types were considered, and the form finally adopted was the "Stoney" Sluice Gate. This is a steel gate, placed between masonry piers, lifting vertically out of the water. All the working parts of this dam are above water, making repairs easy, and the gate cannot become jammed by drift or floating ice. Four openings, of 52½ ft. each, were provided, and the total length of dam, including piers and abutment, is 256 ft. The piers, Fig. 2, are 20 ft. high, with vertical sides. The up-stream end is vertical for 7 ft., and has a batter of 1 on 1 in the next 9 ft., forming an ice breaker; the remainder is vertical. The piers are recessed on each side to form bearings for the gates. From the gate recesses, forward, the piers are almost entirely of cut stone. The nose stones and the down-stream quoins are granite. The remainder of the stone, including the down-stream nose stones and the coping, is oolitic limestone. The body of the piers is concrete, with a facing of granite paving blocks, with headers and stretchers laid alternately.

The foundations are of concrete, and a concrete sill, with top surface of granite paving blocks, 6 in. thick, extends between the piers. This sill is 39 ft. wide. The outer pier is 9 ft. wide and 62 ft. 8¾ in. long. The three intermediate piers are 8 ft. wide and

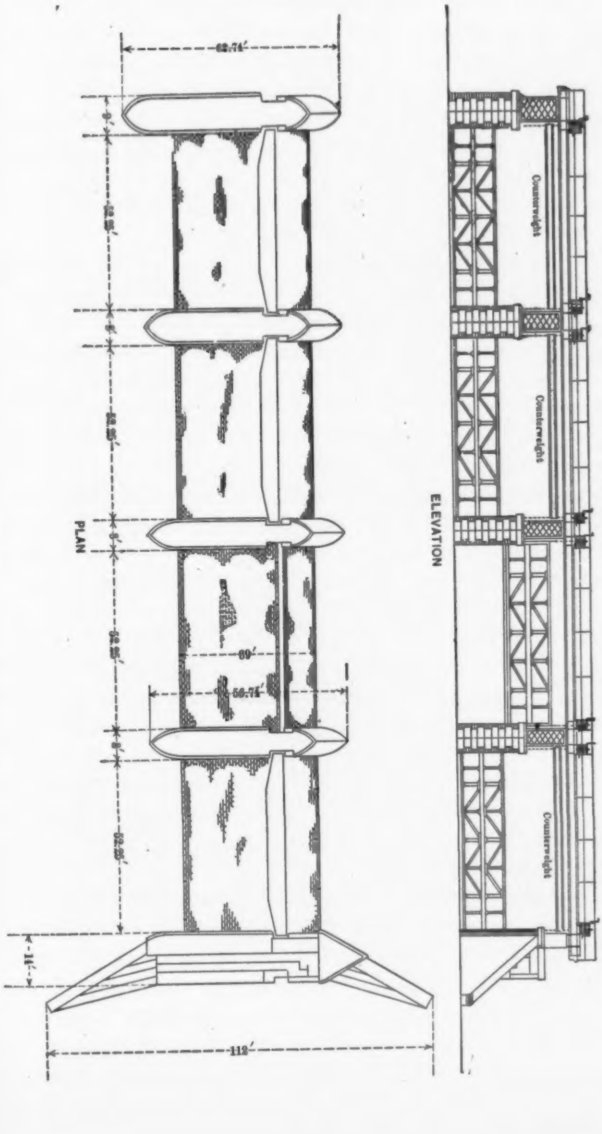

MOVABLE DAM
ELEVATION
PLAN
Counterweight
Counterweight
Counterweight
89'
14'
112'

FIG. 1.

56 ft. 8⅝ in. long. At the in-shore end of the dam there is an abutment with wing-walls. This abutment is 14 ft. wide, reducing by offsets to 6 ft. on top. It is 24 ft. high and 112 ft. long, from end to end of wing-walls. The piers and abutment are covered with coping which projects 6 in. The masonry is of ample dimensions to resist the pressure of water and ice against the closed gates.

The gates, Fig. 3, made entirely of steel, are 54 ft. 3½ in. long (from center to center of end rollers), and are 12 ft. 11½ in. high. At mean high water they will be immersed 12 ft.

The gates have three horizontal ribs, placed at top, bottom and intermediate. They are 5 ft. wide through the center portion, and are reduced to 3 ft. at the ends. The ribs are connected by end plates, by three sets of intermediate cross-frames, by diagonal bracing on the back, and by a sheathing of ⅜-in. steel plates on the face. The gates bear against cast-steel track-plates, bolted in the quoins of the piers. Trains of live rollers are introduced between the gates and their tracks to reduce the friction when the gates are operated. The weight of each gate is 59 400 lb., which is nearly balanced by a counterweight of 57 600 lb. The gates are operated by hand-power winches, mounted on steel towers at each pier. The towers are connected by a light steel bridge, to give access to the machinery. Sprocket chains, attached to the ends of the gate, pass over sprocket wheels driven by the winch gearing, and connect with the counterweight. This counterweight is a horizontal box-girder, loaded with cast-iron weights, and extends the entire length of the gate. Two winches are provided for each gate (one over each end), and are connected by a horizontal shaft, to insure their working together.

The crib dam, Fig. 4, is 240 ft. long, and consists of fifteen rock-filled cribs, of such height that their tops will be 4½ ft. below mean high water. These cribs are 12 ft. wide and 19 ft. long. They are built of logs, 10 in. in diameter at the small end, and notched and drift-bolted together where they overlap. The projecting ends of the logs in one crib interlock with the projecting ends of those in the adjoining cribs. They form the center of a rock fill which is carried to a height of 5½ ft. above mean high water, or 10 ft. above the top of the cribs. This rock fill is level on top for a width of 12 ft., and slopes away on each side to the bottom of the river. The up-stream slope is 1 on 1, and the down-stream slope is 1 on 2.

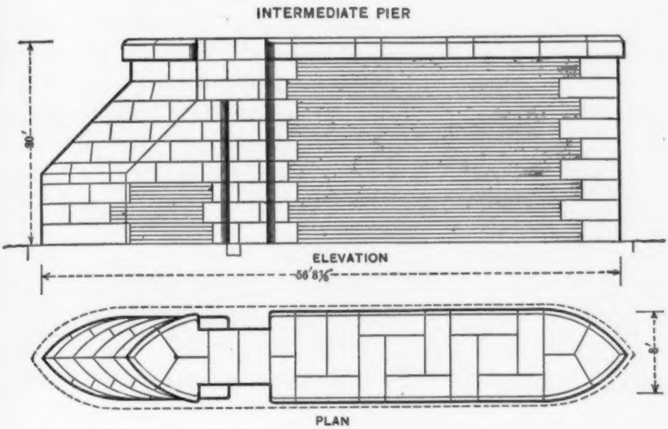

INTERMEDIATE PIER
ELEVATION
56'8½"
PLAN

FIG. 2.

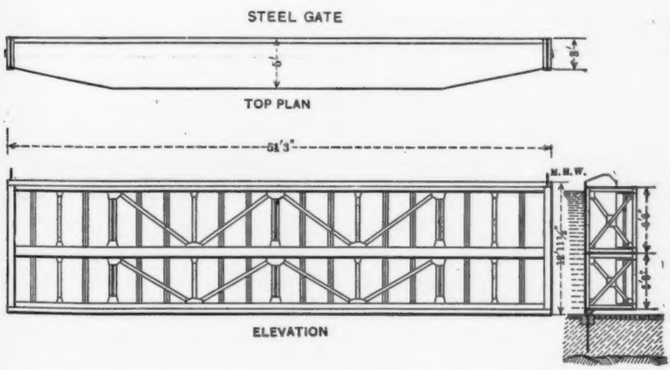

STEEL GATE
TOP PLAN
51'3"
ELEVATION
SECTION

FIG. 3.

The submerged weir, Fig. 5, is 240 ft. long and 20 ft. wide, with its crests 4½ ft. below mean high water. A frame of 12 by 12-in. timber, about 3 ft. high, is anchored to the underlying rock by 8-ft. wedge-bolts, 1½ in. in diameter, and is filled with concrete. The crest of the weir, 3 ft. higher than the sides, is rounding, with a radius of 4 ft. 9 in., and is paved for a width of 5 ft. with granite paving blocks, bonded into the concrete. The upper slope, 1 on 1, and the lower slope, 1 on 4, are tangent to the curve of the crest.

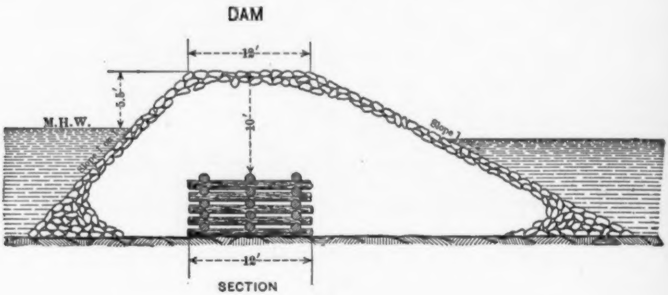


FIG. 4.

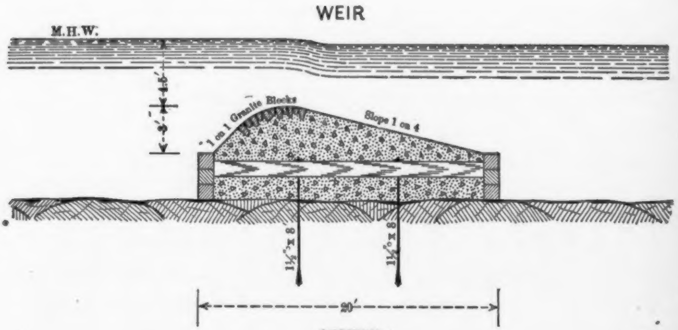


FIG. 5.

H. von Schon, M. Am. Soc. C. E., Chief Engineer of the Michigan Lake Superior Power Company, was in general charge of the water-power development, on the American side, together with the work in the rapids, and the writer was employed as Assistant Engineer, in charge of constructing the compensating works.

FIG. 1.—LAKE SUPERIOR COMPENSATING WORKS. THE BREAKWATER, FROM THE RAILROAD, AUG. 16TH, 1901.

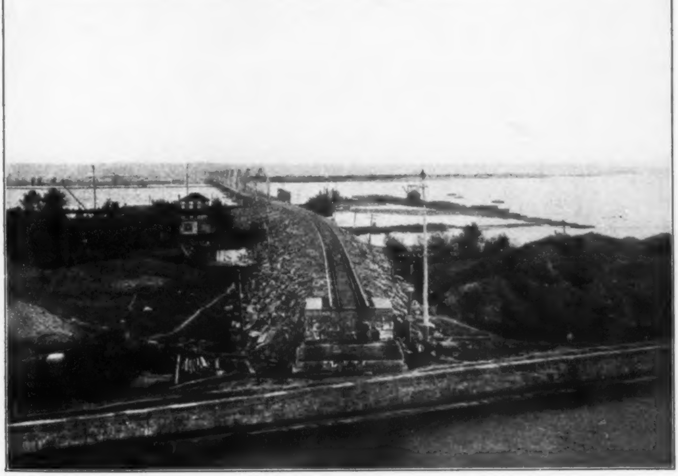


FIG. 2.—LAKE SUPERIOR COMPENSATING WORKS. VIEW FROM CANADIAN CANAL, SEPT. 24TH, 1901.

The sections were to be built one at a time, as it was not desirable to obstruct a large portion of the channel prior to the opening of the water-power canal. The movable dam was to be constructed first and then the submerged weir, the crib dam, and the embankment, in order. This programme was not entirely carried out, however, as delays in the construction of the water-power canal made it desirable to suspend operations in the rapids, after the completion of the movable dam; and, though the embankment was built, the remaining sections were left to a future time, when the quantity of water withdrawn through the canal would make the completion of the regulating works necessary.

Construction was commenced on June 13th, 1901. A cement storehouse, blacksmith shop and other buildings were erected, and the plant was assembled, on Gorby's Island, at the north end of the International Bridge. Through arrangement with the Algoma Central and Hudson Bay Railroad Company (one of the subsidiary organizations of the Lake Superior Power Company), a short spur track was built from the Canadian Pacific Railway, to give access for materials, etc. This track leaves the main line at a high embankment, and, with nearly 180° of 24° curve, drops on a 4% grade down to the level ground at the site of the work. The following plant was used:

Two 60-ft. derricks, with double-drum hoisting engines;
One steel traveling derrick;
One No. 6 gyratory stone crusher, with engine and boiler;
One 4-ft. cube, concrete mixer and engine;
One 1-yd. orange-peel bucket;
Eight 1½-yd. side-dump cars;
Six small platform cars;
Track, 2 000 lin. ft., 30-in. gauge, 25-lb. rail;
Four small barges;
One large sand barge, with 12-in. sand pump, and engine;
One small steam launch;
Two 12-in. centrifugal pumps, with engines and boilers;
One 4-in. pulsometer;
Two steam drills;
Three 20-h-p. boilers;
Skiffs, blacksmith's outfit, small tools, etc.

Tugboats and a dredgeboat were hired, as needed.

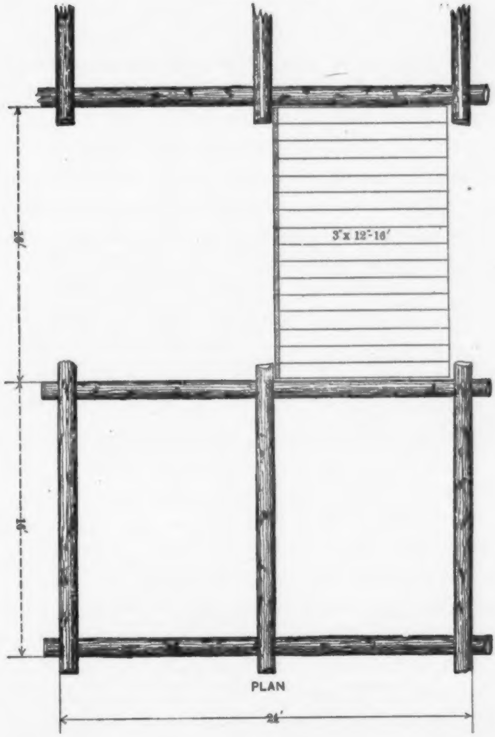
The plans contemplated the construction of the dam inside of a single, large coffer-dam. This necessitated the erection of a breakwater to protect the site, as the river had a velocity of 10 ft. per sec. and was about 12 ft. deep. The construction of a large coffer-dam, in such a current, was not considered practicable, and later experience entirely justified this conclusion. The length of breakwater required to protect the site was 980 ft. At the shore end the water was quite shallow, not more than 2 ft. deep, and it gradually increased to 15 ft., at the outer end. The breakwater was made of stone-filled cribs spaced 16 ft. apart, in the clear. The intervals were left to provide passages for the water, to avoid damming up the river and thereby increasing the current while the cribs were being placed. These openings were afterward filled.

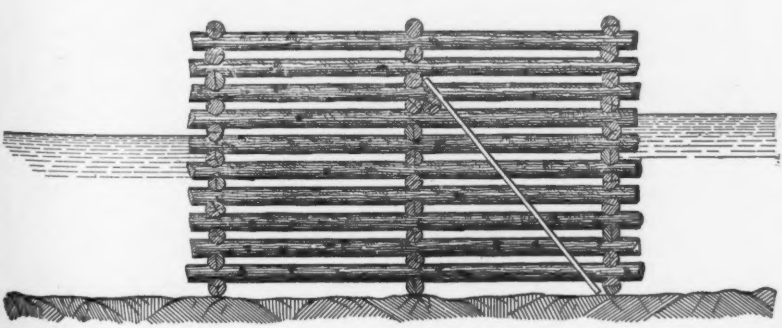
The cribs in shallow water were 16 ft. square, while those in deep water were 16 by 24 ft., and placed with the longest side parallel to the current. They were of spruce logs, from 8 to 10 in. in diameter, fastened at the corners with ¾-in. drift-bolts, and had a bottom of logs on which the stone filling rested. The breakwater was started on July 6th, but was not completed until September, slow progress being made on account of the difficulty in securing sufficient labor. Only lumbermen who were accustomed to logging in swift water could be employed, as this work, on account of its location in the head of the rapids, was particularly hazardous.

The cribs in the shallow water near shore were built in place without much difficulty, but, as the depth and the current increased, a different method had to be adopted. A convenient site was found, about half a mile above the breakwater, where there was little current and ample depth, to build the cribs in the water, from which place, with the aid of a tugboat, they were floated down to the work, one at a time. The river for 2 000 ft. above the breakwater was obstructed by numerous shoal spots, formed of loose rock and boulders brought down by the ice, and though these places were marked by buoys, they were too close to the rapids to risk sending the tug into this region, therefore the cribs were floated down on the end of a line 2 000 ft. long.

Stone for filling the cribs was received on cars at the railroad dock, a mile or more above, where it was loaded on small barges. The tug took a barge in tow, and, with a crib hitched on the front

BREAKWATER





SECTION

Fig. 6.

end of the barge, dropped them with the current, down to the breakwater, maneuvering so as to bring the crib to place, where it was filled with stone. The empty barge was then towed up out of the rapids. First, two anchor cribs were placed, about 400 ft. apart and 1 000 ft. above the breakwater, and then three anchor cribs, on a line parallel to and 250 ft. above the breakwater. These were used to assist in guiding and in holding the floating cribs, because, on account of the many cross-currents, the tug, 2 000 ft. away, could not locate them accurately. As soon as a crib had received enough stone to hold it against the current, the sides were built up to a height of 5 or 6 ft. above the water and it was filled with stone, making it perfectly secure. After all the cribs had been placed, the intervals were closed by 2-in. or 3-in. planking, depending upon the depth of the water.

A log, about 12-in. in diameter, was placed just above the water surface, extending between adjacent cribs, to support the upper end of the sheathing, which was laid with a slope up stream. The lower end of the sheathing rested on the bottom of the river and the current held it firmly in place. This construction is shown in Fig. 6. Where the depth of water exceeded 10 ft., it was found impracticable to close the openings with sheathing, as the planks could not be placed with sufficient accuracy, but were twisted out of the men's hands as they tried to guide them in their descent to the bottom. Several different methods of placing the planking were tried without success, and, in the deeper water, additional cribs were dropped into the intervals, making the outer end of the breakwater solid cribwork. A track, of 18-ft. gauge, was laid on top of the breakwater for the traveling derrick, which was used quite extensively, first for the construction of the breakwater and later for the coffer-dam. The breakwater served its purpose admirably, as there was practically no current below it and there was a difference of about 19 in. in the water surfaces above and below the sheathing.

The following materials were used in the breakwater:

Spruce logs, 38 291 lin. ft.

Lumber, 15 808 ft., B. M.

Drift-bolts (¾ by 20-in.), 8 100 lb.

Sandstone, 4 810 cu. yd.

The river bed, which could be plainly seen through the clear

PLATE XXXVI.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIV, No. 996.
STICKNEY ON
LAKE SUPERIOR COMPENSATING WORKS.

FIG. 1.—LAKE SUPERIOR COMPENSATING WORKS. COFFER-DAM PUMPED OUT. VIEW FROM SHORE, DEC. 10TH, 1901.

FIG 2.—LAKE SUPERIOR COMPENSATING WORKS. COFFER-DAM PUMPED OUT. VIEW LOOKING SOUTHEAST, DEC. 10TH, 1901.

water, was smooth rock, intersected by numerous long cracks, extending in every direction. The greater part of it was swept clean by the current, but about one-half of the area to be inclosed by the coffer-dam was overlaid by a deposit of gravel and boulders which, in places, was from 6 to 7 ft. thick. On September 3d, a small dipper dredge and a dump-scow were placed behind the breakwater to remove this deposit, preparatory to the construction of the coffer-dam. Difficulties were experienced in disposing of the dredged material, as the scow could not be handled satisfactorily within the limited area sheltered from the current, and it was decided to clean the bottom along the line of the coffer-dam only, leaving the bulk of the material to be taken out later. The use of the scow was discontinued and the dredged material was cast to one side. By this

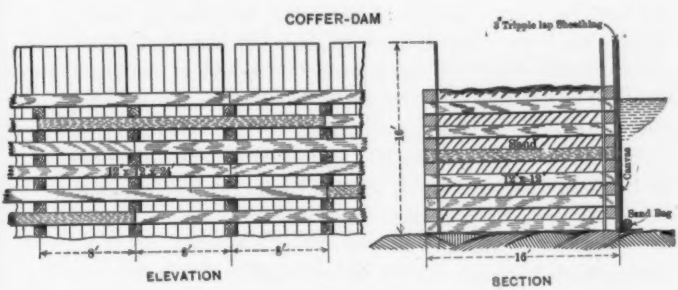


FIG. 7.

method, a dike was formed, just north of where the coffer-dam would rest, extending 3 or 4 ft. above the water surface, and making an additional protection on that side. This work being completed, the dredge and the scow were removed.

The coffer-dam, Fig. 7, was in the shape of a trapezoid, inclosing an area of 29 120 sq. ft., and measuring 900 ft. around the outside. Each of the four sides was a continuous crib of 12 by 12-in. timber, 16 ft. wide and 12 ft. high, built in place in the water. These cribs were settled to the bottom, in their proper locations, and were then connected at the corners of the dam. The cribs were sheathed inside with 1-in. plank, placed vertically, and battens were nailed over all cracks, making tight joints. Between the two walls, thus formed, sand filling was placed, to give the necessary weight to the

dam. A row of tongued and grooved sheathing, made with three thicknesses of 1-in. plank nailed together, was placed around the outside of the cribs. The lower end of this sheathing was sharpened to a chisel edge, and each piece was well driven with a wooden maul, so as to broom up the lower end and make a close fit against the rock bottom. The sheathing was nailed at the top and near the bottom, a diver being employed in the operation. A strip of canvas, 5 ft. wide, was laid all around the bottom of the dam, extending up the sides for 3 ft. and out on the river bottom for 2 ft. Battens were nailed over the canvas, to hold it against the dam, and sand bags held the part in place on the bottom. A second 5-ft. strip of canvas encircled the dam above the first, overlapping it a few inches. The only purpose served by a sand filling was to give enough weight for stability, while the tongued and grooved sheathing and the canvas were depended upon to make the dam water-tight. Sand was pumped from the lake bed, near Point aux Pins, about 8 miles away, and was delivered on barges at the breakwater, where it was loaded into skips and handled by the traveling derrick either directly into the coffer-dam or into small dump-cars on top of the dam. Track was laid on the cribs, extending around the entire dam, with a turntable at each corner, so that the cars could reach all parts of the dam. The coffer-dam was commenced on September 25th and was completed on November 30th. The following materials were used in its construction:

Timber (12 by 12-in.), 264 576 ft., B. M.
Planking (1-in.), 92 252 ft., B. M.
Drift-bolts (¾ by 22-in.), 5 620 lb.
Sand, 4 986 cu. yd.
Canvas (60 in. wide), 918 yd.

A clay puddle, between the walls, would have made the triple-lap sheathing and the canvas unnecessary, but clay was not to be found in the immediate vicinity, and could not have been used, except at considerable cost, both for placing and removing it.

The pumping plant, consisting of two 12-in. centrifugal pumps, with direct-connected engines, and three 30-h-p. boilers, was installed on a small barge, which was moored at the lower side of the coffer-dam. A 12-in. rubber suction hose, 40 ft. long, led from each

pump across the top of the coffer-dam, and 12-in. discharge pipes carried the water over the side of the barge. Each pump had a capacity of 4 000 gal. per min., with a lift of 20 ft., which was the height the water was raised. A house was built on the barge, covering and protecting the machinery from the weather. By the time the coffer-dam was completed, cold weather had set in, and there was a 6-in. sheet of ice on the water inside the dam.

It took considerable time to pump out the dam, and when the bottom was finally exposed, it was found that several seams in the rock were spouting water. These seams were caulked with oakum, or with wedges of soft wood, which reduced the leakage very considerably, so that it could be handled easily with one pump. A sump, about 5 ft. deep, was blasted out of the rock where the suction pipes were located, and drains to conduct the water were made close to the inner wall of the coffer-dam.

The coffer-dam proved entirely satisfactory, but, on several occasions, there was some little trouble with the pumps, usually due to the neglect of the men in charge of them. On one particularly cold night, the coffer-dam was flooded to a depth of about 2 ft. by a rather unusual occurrence. The discharge pipes from the pumps projected through the house, and were not protected from the weather. They did not extend quite to the side of the barge, and, if the pump was run very slowly, some water fell on the deck, where it froze. There was always more or less ice on the deck, but it was chopped away from time to time and not allowed to accumulate. On the night in question, the pump runner did not venture out of the house, as the temperature was about 20° below zero, and the ice formed more rapidly than usual. Ice built up, in a column, from the deck to the mouth of the pipe (5 ft.) and then formed in the pipe, gradually reducing the outlet until the discharge was filled with solid ice as far back as the pump, a distance of 16 ft. As the pump kept on revolving, some little time elapsed before the discovery was made that it was not throwing water. As soon as the facts were known the second pump was put in operation and, in a short time, cleared the coffer-dam.

On December 7th was commenced the excavation of the gravel and boulders which had been left inside the line of the dam. The traveling derrick had been shifted from the breakwater to the north end of the coffer-dam, and was used in handling this material.

Tracks were laid inside the dam for small platform cars, which carried skips of about 1 cu. yd. capacity. These were loaded by hand and pushed up within reach of the derrick, which hoisted them to the top of the dam and dropped the material into small dump cars. These cars were run ashore, over a trestle, and the gravel, etc., was used in filling a part of the river, beyond the end of the International Bridge. An area of about 1 acre was thus reclaimed, and this ground was used, later, for storing structural steel for the gates. On excavating the rock, below the level of the drains, water commenced to collect in the workings, coming through the cracks and seams. Bailing and pumping with hand-pumps served, at first, in keeping the excavation clear, but, as the foundations extended and the seams opened up more and more by the blasting, the quantity of water to be handled increased. Ice formed very rapidly and was nearly as troublesome to excavate as rock.

The working force became much reduced, on account of the low temperature, and it was impossible to get men to work at night. In this contingency it was decided to erect buildings over the site and to establish a heating plant in them, so that the work could be carried on regardless of the weather. Five buildings, each 20 ft. wide, 30 ft. high, to the eaves, and of sufficient length to cover the piers and the abutment, were constructed. Between these buildings, sheds, 44 ft. wide, 40 ft. long and from 8 to 12 ft. high, were erected, the whole being connected as one building and sheltering all parts of the work. The buildings over piers were located so as to leave room for a track alongside the pier. Numerous doorways were made, and a system of tracks was laid so that materials could be taken on cars to all parts of the buildings. The buildings were covered with tar-paper, in order to retain the heat. The heating plant consisted of three 20-h-p. boilers (taken from hoisting engines) and ten radiators, made with 1 900 lin. ft. of 2-in. steam pipe, in coils. These coils were distributed around the walls of the buildings, and, with the ells, nipples, etc., gave about 1 125 sq. ft. of radiating surface. The area of the walls and roof was 41 400 sq. ft. and the interior volume of the buildings about 280 000 cu. ft. Steam was carried at a pressure of 90 lb., and no difficulty was found in maintaining a temperature above freezing, even with the outside temperature 20° below zero. The usual temperature was about 40° fahr. Numerous valves were placed in the heating coils, so that

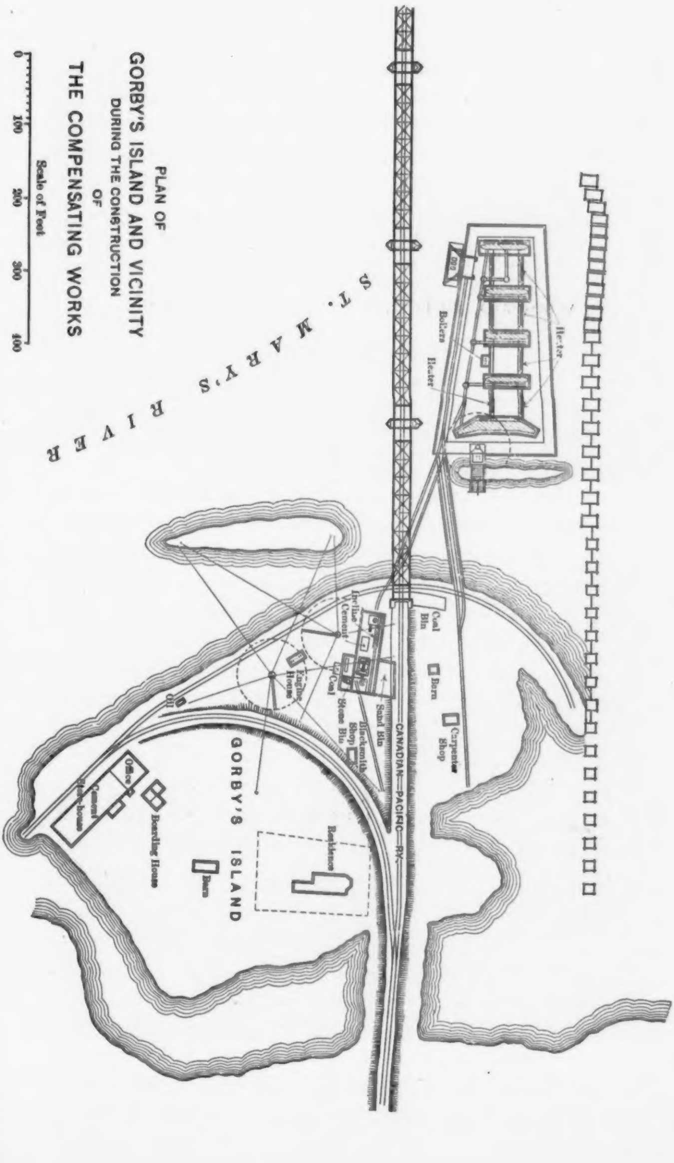

PLAN OF
GORBY'S ISLAND AND VICINITY
DURING THE CONSTRUCTION
OF
THE COMPENSATING WORKS
Scale of Feet
0
100
200
300
400
ST. MARY'S RIVER
GORBY'S ISLAND
CANADIAN—PACIFIC—RY.
Boilers
Heater
Coal Bin
Barn
Carpenter Shop
Sand Bin
Blacksmith Shop
Stone Bin
Coal
Engine House
Cement
Oil
Office
Boarding House
Residence
Barn

FIG. 8.

connections could be made, and steam obtained, for running drills, pumps, etc., wherever needed. Light was provided by sixty incandescent electric lights, each of 16 c-p. The Tagona Water and Light Company, of Sault Ste. Marie, Ont., furnished current for these lights. The buildings were commenced on January 4th and finished on January 16th, the following materials being used in their construction:

Lumber, 88 576 ft., B. M.;
Laths (4 ft.), 12 200;
Tar-paper, 256 rolls;
Wire-nails, 20 kegs.

Fig. 8 is a general plan of the work, and shows the location of the buildings. The excavating was recommenced and continued both night and day, as there was now no trouble in getting labor. The water in the excavation was taken care of by a 4-in. pulsometer taking its steam from the heating coils.

At first, no blasting was allowed, for fear of damaging the buildings, and the rock was loosened by picks and wedges. After a few experiments, however, it was found that light charges of dynamite would not cause material injury, and blasting was again commenced. The rock was an argillaceous sandstone, which appeared to be pretty solid, but it was found to lie in strata varying in thickness from 6 to 15 in. These layers were separated by beds of hard argillaceous material, which disintegrated rapidly when exposed to the air, and became quite soft when wet. The pier foundations are from 6 to 10 ft. deep, below the original bed of the river, and the excavation for the sill between the piers varies in depth from 4 to 7 ft. Along the up-stream edge of the sill, a trench, extending to the bottom of the pier foundations, was made, to furnish drainage for the excavations. This trench was filled with concrete when the sill was laid, and forms a cut-off wall to prevent any leakage under the sill. The materials removed were:

Gravel and boulders....................	800 cu. yd.
Sandstone	2 708 " "
Total	3 508 cu. yd.

The concrete plant was located in a large building at the north end of the railroad bridge, alongside the tracks of the Canadian

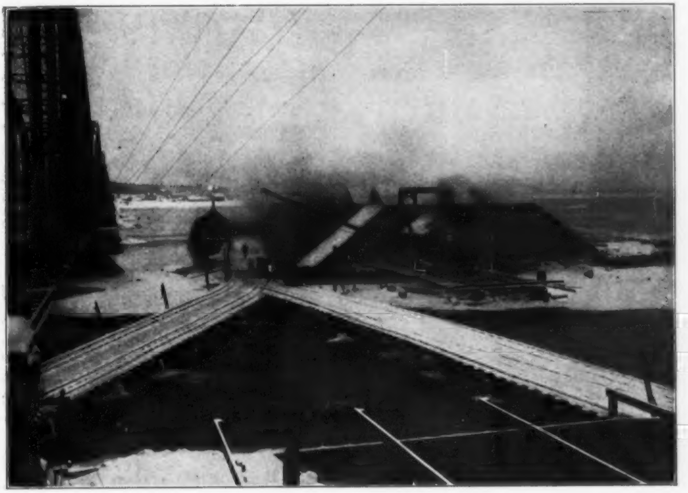


FIG. 1.—LAKE SUPERIOR COMPENSATING WORKS. VIEW FROM SHORE, FEB. 1ST, 1902.

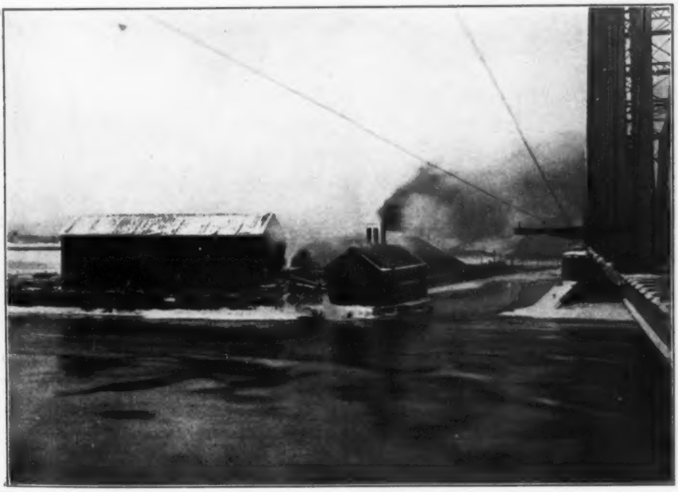


FIG. 2.—LAKE SUPERIOR COMPENSATING WORKS. VIEW FROM RAILROAD BRIDGE, FEB. 1ST, 1902.

Pacific Railway. It consisted of a No. 6 gyratory stone-crusher and a 4-ft. cube, concrete-mixer, both of which were operated by steam furnished by a 60-h-p. boiler. The stone-crusher was set up at one end of the building and the concrete-mixer at the other. Between them there was an elevated bin, of about 100 cu. yd. capacity, for the storage of crushed stone. A belt-elevator took the broken stone from the crusher up into a rotary screen at the top of the building, where the dust was removed, and the stone dropped into the bin.

Opposite the stone bin there was a sand bin, of about 600 cu. yd. capacity, built on the slope of the railroad embankment, the latter being about 20 ft. high. A track, extending between the two bins, led up an incline to an elevated platform, in which was placed the charging hopper over the mixer. A small dump-car was used to deliver the stone and sand at the mixer, being hauled up the incline by a power hoist; and cement, in sacks, was carried to the charging platform by a belt-elevator. The car was divided by a partition into two compartments, of such size as required to measure the proper quantities of sand and stone for a batch of concrete. Sand was shoveled into the car, which was then moved to the stone bin, where a gate was opened, letting the stone drop into the car.

The sand and stone for the crusher were brought to the work on cars, and were unloaded from the main line of the Canadian Pacific Railway, the sand being shoveled directly into the sand bin, and the stone thrown down the embankment, where a pile of about 3 000 cu. yd. was accumulated.

A track led from the crusher to the stone pile, so that the stone could be loaded into a small car and dumped directly into the crusher. Steam pipes were laid through the sand and stone bins and also into the water tank, so that all the concrete materials could be warmed before mixing. A track, under the mixer, led out to the coffer-dam, over a trestle, for the delivery of concrete. Wooden boxes, with bottom doors, were used in conveying concrete. These were placed on platform cars, which ran under the mixer and received a charge. They were then run out to the coffer-dam, where a derrick picked up the box and placed it on a second car, down inside the coffer-dam. The pier foundations and the sill were divided by wooden bulkheads into twenty-four sections, each containing approximately 100 cu. yd., as this was the quantity of concrete that could be placed conveniently in one day. This concrete was

mixed in the proportions, 1 part cement, 1½ parts sand and 4 parts broken stone. The water coming into the excavation required almost constant pumping, and efforts to stop up the leaks only resulted in their breaking out in new places in a short time. Various expedients were adopted to stop or divert the flow at the point where concrete was being placed. The smaller leaks were caulked with oakum, and, if this was not sufficient, a strip of canvas was hung along the face of the excavation and the concrete was placed in front of it, confining the water between the canvas and the natural rock, where it would flow until it escaped into the next section. After the concrete was raised above the leak, it would be shut off altogether. At one point, the inflow had increased to such extent that a 3-in. pipe was required to carry the water. Here a small chamber was hollowed out of the rock, to form a reservoir, into which the end of the pipe was inserted. The opening was walled up with brick, laid in cement mortar, and the water was allowed to flow through the pipe, while concrete was placed, inclosing the pipe. After allowing sufficient time for the concrete to set, the pipe was plugged.

The first concrete was placed on February 4th, and on March 3d the foundations of the piers and the sill, containing 2 113 cu. yd., were completed. The granite paving on the sill was completed two days later, having been laid as soon as the concrete had set. This paving covered an area of 880 sq. yd. The blocks averaged about 4½ by 6 by 9½ in., and laid 26 to the square yard.

Trestles were erected at each pier, to support a traveler, used in building the pier. These trestles extended the full length and width of the buildings, so that the traveler spanned the pier and the material track, alongside. The travelers were constructed as follows: A pair of trucks, each with two double-flanged wheels, was connected by two bridge pieces, on which the dolly moved. The dolly was a small platform car, carrying a hand-power winch. The center of the car platform was removed, so as to permit the hoisting line from the winch to pass through the frame.

The travelers were moved, longitudinally and transversely, by using pinch-bars under the wheels. The stone for the piers was on the ground long before the foundations were completed.

Although this stone had been quarried late in the fall, and was then stacked where it was exposed to extreme cold weather, only one

FIG. 1.—LAKE SUPERIOR COMPENSATING WORKS. ABUTMENT PIER, APR. 21ST, 1902.

FIG. 2.—LAKE SUPERIOR COMPENSATING WORKS. ERECTING STEEL, APR. 21ST, 1902.

piece was lost through the action of frost. It was necessary to warm all materials before placing them in the piers, and this was done by placing them in the buildings from 18 to 24 hours before they were needed. All the dimension stone and the facing blocks, required for one course, were placed on the track, alongside of each pier. On several occasions, when it was desired to warm the stone quickly, this was done by pouring hot water over them, the water being heated, in buckets, by steam from the heating coils. The dimension stones of a course were set, and then three courses of the facing blocks were laid. Five courses of the facing blocks were equal in height to one course of dimension stone. The facing blocks averaged 4¼ by 5½ by 8½ in., and laid 34 to the square yard. They were somewhat smaller than the paving blocks, and were more regular in shape and more uniform in size. The concrete backing, mixed in the proportions, 1 part cement, 2 parts sand and 5 parts broken stone, was placed as high as the facing, and then two more courses of granite blocks were laid. As no forms were used to support the facing, it was necessary to allow a certain time, usually 12 hours, for the mortar in the joints to set, before the concrete was placed, otherwise the facing would be crowded out of line. The masonry averaged 1 ft. in height, on each pier, per day, except on the abutment, where delays were occasioned by a change in plan, requiring additional stone from the quarry. The first stone was set on March 1st and the abutment was completed on April 10th. The piers and abutment contain the following classes and quantities of masonry:

Dimension stone	502.33 cu. yd.
Granite facing	150.67 " "
Concrete backing	1 252.00 " "
Total	1 905.00 cu. yd.

The materials used in the piers and sill are as follows:

Cut granite, 132.78 cu. yd.;
Cut limestone, 369.55 cu. yd.;
Granite facing blocks, 26 500;
Granite paving blocks, 23 000;
Portland cement, 7 039 bbl.;
Sand, 1 477 cu. yd.;
Broken stone, 3 283 cu. yd.

The granite was obtained from St. Cloud, Minn., and the limestone came from Southern Indiana. The stone, for concrete, was quarried about a mile from the work. The cements used were "Vulcanite" and "Alpha."

The heating of buildings was discontinued on March 28th, the weather having moderated so that it was no longer necessary.

During the 72 days that the plant was in operation 292 tons of coal were consumed. The removal of buildings was immediately commenced, and was completed on April 5th. There now remained, to complete the substructure, only the setting of the roller tracks in the quoins of the piers. This proved to be a very tedious operation, on account of the large number of bolt holes drilled in the granite.

The track castings are 12 in. wide, 4 in. thick and 20 ft. long, made in two pieces, and are fastened to the masonry by two sets of bolts crossing each other at right angles, as shown in Fig. 9. The bolts had to be placed with great care, in order that the casting would slip over them, as there was very little clearance between bolt and plate. Wooden templates were made, to locate the holes, and the drilling was done by hand. The bolts, parallel to the axis of pier, were set first with sulphur, being centered with small wooden wedges and a swallow's nest of clay, placed so as to make the hot sulphur run back into the hole. The track-plates were slid over the bolts, and pulled back tight into the quoin. The transverse bolts had to be inserted through the track-plates and fastened into the holes previously drilled for them. These bolts were made with a ½-in. hole bored longitudinally through the center. They were fastened to the masonry with cement grout, pumped through the center hole, and filling the annular space around the bolt.

All materials were removed from the coffer-dam, and the pumps were stopped on April 23d, having run continuously for a period of 145 days, with an expenditure of about 595 tons of coal.

The gates were put in place by the Dominion Bridge Company, of Montreal, Que., that company having been awarded the contract to furnish and erect the gates complete, on April 12th, 1901.

The fabrication of structural steel, etc., had been completed early in the winter, and the greater part of this material arrived

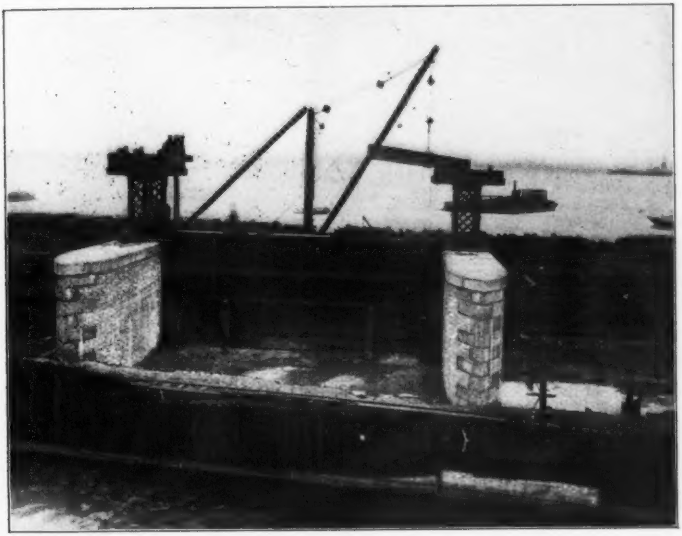


FIG. 1.—LAKE SUPERIOR COMPENSATING WORKS. STEEL GATE IN PLACE, APR. 21ST, 1902.

FIG. 2.—LAKE SUPERIOR COMPENSATING WORKS. NORTH VIEW OF END.

during February, 1902, being unloaded close to the work, to await the completion of the substructure.

On April 4th, the Bridge Company began preparations for the erection. A traveling derrick, with a small air-compressor (to operate the riveters), was installed on the west wall of the coffer-dam; tracks were laid to convey the materials from the shore; and the materials were sorted and distributed within reach of the derrick.

Four trestle-bents were constructed at each opening, to support

SECTION THROUGH QUOIN

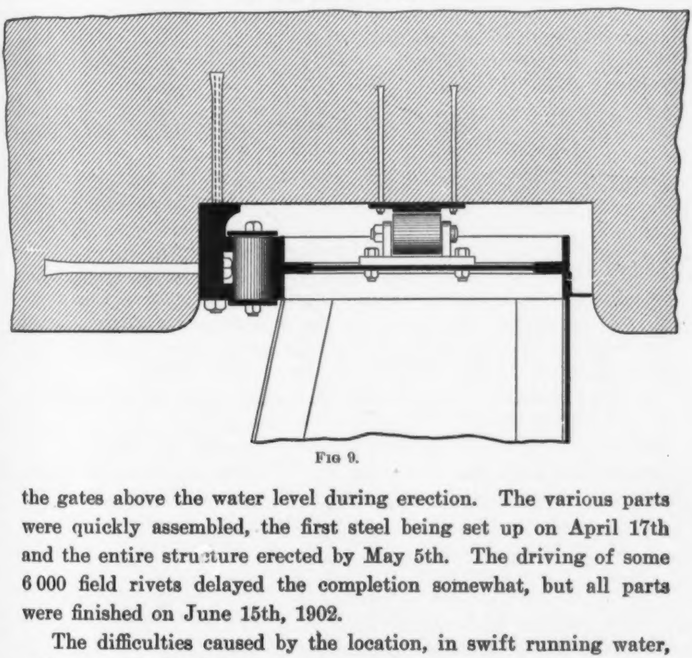


FIG 9.

the gates above the water level during erection. The various parts were quickly assembled, the first steel being set up on April 17th and the entire structure erected by May 5th. The driving of some 6 000 field rivets delayed the completion somewhat, but all parts were finished on June 15th, 1902.

The difficulties caused by the location, in swift running water, and the methods adopted for constructing masonry during extreme cold weather, made necessary a large amount of preparatory work before actual construction commenced, and added materially to the interest in the work.

DISCUSSION.

Mr. Le Conte.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The author of this most interesting paper apparently invites discussion on the compensation devices adopted, but, neverthelesss, does not furnish the data necessary for a complete discussion, thus leaving much to mere assumption.

There are eight bridge spans furnishing areas for discharge, hence the average discharge at low stages must approximate 7 000 cu. ft. per second for each span; but, from other sources of information it is found that the separate discharges at each span at low stage would be more approximately as follows:

Canadian Side—	Span	1....	2 550 cu. ft. per second.
	"	2....	10 070 " " "
	"	3....	11 670 " " "
	"	4....	10 950 " " "
	"	5....	10 940 " " "
	"	6....	6 820 " " "
	"	7....	1 790 " " "
American Side—	"	8....	1 367 " " "
Total discharge.........			56 157 " " "

It is clear, of course, that the greatest public danger to be apprehended is the bare possibility of lowering the lake level below the minimum record. In considering such problems, it is always safer to consider the very worst features that could possibly arise; because the worst conditions have a faculty of always happening at the very time when one is least prepared to meet them.

If it be assumed that the four sections are actually built, as suggested by the author, then there will probably be the following volumes for discharge purposes at low stages of the lake:

Span 1.	Embankment....	0	cu. ft. per second.
" 2.	Closed sluices....	0	" " "
" 3.	High dam.......	0	" " "
" 4.	Submerged weir..	2 170	" " "
" 5.	Unobstructed flow	10 940	" " "
" 6.	" "	6 820	" " "
" 7.	" "	1 790	" " "
" 8.	" "	1 367	" " "

Total flow........23 097 cu. ft. per second, to be deducted from 56 157 " " " minimum flow,

leaving 33 070 " " " to pass down through the canal to the power-house.

This quantity seems to be uncomfortably close to the actual requirements, 32 000 cu. ft. per second, for power purposes. Mr. Le Conte.

In view of the many uncertainties connected with these calculations, such as high wind in combination with "big high" barometer at the east end of the lake, or a "big low" barometer at the west end of the lake, all of which are powerful, yet largely unknown, quantities tending to lower the low-water plane below the present record; it would seem, under the circumstances, far more advisable to put in another span of sluices. This would certainly give more complete command of the outflow at the head of the rapids at times when it is most needed.

The writer would be pleased to know the reasons for selecting the site of the regulation works above instead of below the bridge.

The difficulties met in contending with the swift current, and the manner in which they were finally overcome, are very instructive and highly interesting. The writer has had similar experience, and in many cases has found it more advisable, in many respects, to build the cribs in the form of an equilateral triangle in plan with the point up stream, as it was found that they could be handled much more easily in a swift current. The plan usually adopted in freezing weather is to shut down and quit work for the season, hence the author's description, of how they housed in the work on the sluices and heated the enclosure and the materials artificially, so that the men were able to live comfortably under cover and do good work, is certainly very instructive.

The writer was somewhat surprised to note the proportions adopted for concrete in the foundation work. It seemed to him that, for a foundation course, approximately 240 by 40 by 7 ft. thick, it would have been more advisable to use rubble masonry laid with 4 to 1 mortar, the top course of paving stones, of course, being laid and pointed with 2 to 1 mortar. In the same way, the backing for the masonry in the piers could just as well have been made of rubble masonry with 3 to 1 mortar, the facing stones being laid and pointed with 2 to 1 mortar. However, these are all small matters, which do not detract in the least from the value of this highly commendable paper.

G. F. Stickney, Assoc. M. Am. Soc. C. E. (by letter).—In presenting this paper, it was not intended to discuss the compensating devices adopted, but rather to describe the methods used during construction, as that was the part of the subject in which the writer was chiefly interested. The general scheme had been adopted and the plans made before his connection with the work began. At the present time the data necessary for such a discussion are not available. The plans, as originally prepared, contemplated two sections of movable dam, containing four gates each. As a temporary ex- Mr. Stickney.

Mr. Stickney. pedient, the rock fill was substituted for one of these sections, with the intention of replacing it later, when circumstances should require, by a set of gates.

The site selected for the compensating works, on a line 150 ft. above the center of the International Bridge, is practically at the head of the rapids, the actual crest being an irregular line, partly above and partly below the bridge.

The cribs used in constructing the breakwater were large, and floated deep in the water, presenting considerable surface to the current, which made them difficult to handle. There was a strong cross-current sweeping along the face and around the end of the breakwater, caused by the cribs already in place, which increased in force as each successive crib was added. When the floating crib reached the site of the breakwater it was drawn in against this cross-current and held until filled with stone. It is thought that triangular cribs could not have been handled any more easily, under the circumstances, and their shape would not have been as well suited to the construction as rectangular cribs.

The use of rubble masonry for the foundation course and for backing in the piers was not considered, as concrete seemed to be so much better adapted for these purposes, both as regards cost and the facility with which it could be placed, under almost all circumstances. It was desired to make the foundations sufficiently watertight, so that there would be no chance of seepage under the dam, and it is believed that the concrete used will be more effective in that way than rubble masonry laid with 1 to 4 mortar. All stone in the piers, and the paving blocks on the sill, were laid with 1 to 2 mortar.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 997.

THE STRUCTURAL DESIGN OF BUILDINGS.*

By C. C. Schneider, M. Am. Soc. C. E.

With Discussion by

Messrs. W. B. W. Howe, Charles Worthington, J. R. Worcester, Joseph H. O'Brien, Henry B. Seaman, Augustus Smith, R. D. Coombs, Jr., F. T. Llewellyn, Theodore Cooper, Henry W. Post, Gunvald Aus, J. K. Freitag Virgil H. Hewes, L. J. Johnson, H. P. Macdonald, E. P. Goodrich, M. S. Ketchum, George H. Blakeley, John B. Clermont, Oscar Lowinson, Eugene W. Stern, Charles G. Darrach, E. C. Shankland, Foster Crowell, St. John Clarke, William W. Crehore, C. A. P. Turner and C. C. Schneider.

The object of this paper is to submit a set of specifications, for the structural work of buildings, for discussion and criticism.

As this subject has never been brought before this Society, it is expected that an exhaustive discussion will bring out some valuable suggestions from those who have had experience in building construction, and that this may result finally in a more uniform practice as well as in more uniformity in that portion of building ordinances relating to structural work.

These specifications were prepared originally for the instruction and guidance of the engineers employed in the various offices of the company with which the writer is connected. They were to be used not only in places where building laws do not exist, but also to

* Presented at the meeting of October 19th, 1904.

supplement those local building laws which do not give sufficient data.

Since then the writer has made changes and revisions, which, in some instances, might be regarded as departures from the usual practice, but, beyond this, it has been his aim to select what he considers the best practice of the present day.

These specifications are intended to cover only the structural features of buildings of the modern type, in which steel forms a part of the construction, such as would come naturally under the supervision of an engineer, and, therefore, they are not intended for the building trade, but for the use of educated engineers.

A proper and timely subject, to be included in specifications for structural work of buildings, is that of steel-concrete construction, of which a great deal has been used in later years in fire-proof buildings, etc. However, as this Society has recently appointed a committee to investigate and report on this subject, it was deemed advisable to omit steel-concrete construction from these specifications until the committee's investigations have given additional light on the subject.

The Appendix to this paper contains, in tabulated form, extracts from the various building laws which the writer has been able to obtain up to the present time. It will be noticed that the most striking feature in these building laws is their lack of uniformity as to the specified live load. The minimum live loads per square foot prescribed for floors of dwellings, hotels and apartment houses vary from 40 to 75 lb.; for floors of office buildings, from 60 to 150 lb.; for public assembly rooms, churches and theaters, from 80 to 150 lb.; for schools, from 75 to 150 lb., etc. To make the variation still greater, some building laws allow a reduction for columns and foundations on the permissible live loads specified for floors; others do not.

A similar variation also exists in the permissible unit strains allowed for different kinds of material.

In order that the specifications may be understood properly, it will be necessary to explain some of the more important clauses, and give the reasons which led the writer to adopt them.

Loads.

Dead Load.—The dead load, or the weight of the structure itself, including permanent fixtures, can be ascertained easily by careful computation, and is a permanent and reliable quantity.

Live Load.—Attention has been called to the great difference in the live loads specified by the various building laws for buildings to be used for the same purpose, the differences being in some cases more than 100 per cent. These differences should be harmonized, and a more rational method of loading devised, which would produce a structure of ample strength, more particularly in its details and connections, without waste of material in places where it is not needed.

The writer's attention was first called by Theodore Cooper, M. Am. Soc. C. E., to the irrational practice of specifying a uniform live load per square foot; he thought the specified live loads should be a little more than mere guesswork. Since then the writer has been working in accordance with these suggestions, following the lines which have been recognized for years by engineers in specifying live loads for bridges.

The possible maximum superimposed or live loads on buildings for special purposes, such as warehouses or stores for particular kinds of goods, power-houses, department stores, etc., after their interior arrangement has been decided upon, can be accurately determined.

However, there are classes of buildings, the rooms of which may be occupied for various purposes at various times, such as office buildings, stores, hotels, apartment-houses, dwelling-houses, etc. Dwelling-houses are sometimes used for offices, and rooms in office buildings for light manufacturing purposes.

While it is impossible to foresee and provide for all possible contingencies, it is within the limits of possibility to provide for the varying conditions of loading which may occur in a building if used for the purpose for which it was intended.

Live Loads on Floors.—Mr. C. H. Blackall states*:

"The writer has repeatedly counted the number of persons in the various portions of theatres and music-halls, without once finding, even in crowded aisles and standing-room, an average of more than 40 or 50 lb. per sq. ft. extended over more than a few square feet."

This agrees also with the writer's observations.

A live load of 40 lb. per sq. ft., therefore, may be considered the maximum load to be provided for as a distributed load for all floors on which crowds of people may be expected to congregate, such as all kinds of rooms in dwelling-houses, apartment-houses, hotels, office

* *American Architect*, August 26th, 1893.

buildings, schools, churches, theatres, concert halls, ballrooms, drill-rooms, etc.

Mr. Blackall, in conjunction with Mr. A. G. Everett, made a thorough investigation of the actual existing live loads of three office buildings in Boston. These loads were obtained by taking the actual weights of the furniture and contents and the greatest number of people known to be at any one time in an office, the average weight of one person being estimated at 150 lb. The greatest load was found in one of the offices of the Ames Building, amounting to 40.2 lb. per sq. ft.

In only 12.4% of the offices was the floor load in excess of 25 lb. per sq. ft., and in only 26% did it exceed 20 lb. per sq. ft. The greatest maximum average for all floors of any one of the three buildings was 17 lb. per sq. ft.

In accordance with these data, it may be considered safe to assume that a distributed live load of 40 lb. per sq. ft. will be sufficient to provide for a crowd of people as well as for the ordinary loads carried on floors used for offices or similar purposes.

The writer has investigated this subject and endeavored to discover extreme cases in order to find a method of concentrated loading to cover the same.

For this purpose, weights of all kinds of furniture were collected and their contents estimated. The weights were not taken as they actually existed, but as they would be if completely filled with the material for which they were intended.

It was found that the ordinary furniture, such as desks, tables, wardrobes, counters, chests, small safes, etc., may be discarded for extreme loads. The heaviest concentrated loads found in any office were safes.

The portable safes used in offices rarely ever weigh more than 5 000 lb. with contents. This load may be carried by one beam, and, as a safe of this weight is likely to be placed in any office, every floor joist should be calculated for a concentrated load of 5 000 lb. in any position.

The maximum weight of safes generally used in dwelling-houses is 2 000 lb.

The heaviest portable safe manufactured weighs 16 000 lb. and occupies a floor space of 69 by 45 in.

Safes of such excessive weight, however, are not placed on floors

of office buildings in which no special provisions are made for them, unless arrangements are made to distribute the load over at least several beams.

In offices, the weights of all other furniture with contents do not approach that of safes.

Only a few cases of combinations of extreme loads were found to produce results similar to that of a concentrated load of 5 000 lb. They were as follows:

In a large room used as an engineering office, a number of cases with drawers holding drawings were placed in a double row, back to back, in the middle of the room, and used as a table on which to spread drawings. These cases were 31 in. wide and 36 in. high, weighing when completely filled 160 lb. per lin. ft., or both together 320 lb. per lin. ft.; but as their total width was 62 in., they may be considered as being carried by two beams.

A case of drawers for drawings, 31 by 44 in. and 5 ft. high, if completely filled, would weigh 1 200 lb. As there is a possibility of having a whole row of such cases placed along a partition, this would give a load of 326 lb. per lin. ft., which may extend the whole length of a beam.

The weight of a "Wernicke" bookcase about 6½ ft. high was found to be 170 lb. per lin. ft. when completely filled with books.

A row of bookcases might be placed on each side of a partition for the whole length of the room, in which case the load would be 340 lb. per lin. ft. If the partition, instead of running parallel to the beams, should be placed at right angles to them, and if the beams were spaced at the usual distance of 5 or 6 ft. apart, the concentrated load would be only from 1 700 to 2 040 lb. on each beam.

These investigations appear to indicate that a concentrated load of 5 000 lb. on any point of a beam, and a uniform load of about 340 lb. per lin. ft. of beam, will probably cover all possibilities of extreme loading of floors used for office purposes.

A concentrated load of 5 000 lb. is equivalent to the following uniform loads per linear foot of beam of different spans:

Spans........................	10	20	30	40 ft.
Uniform load per linear foot....	1 000	500	333	250 lb.

If the span of the beam is 30 ft. and more, then the load of 340 lb. per lin. ft. would govern. However, as offices are rarely more than 30 ft. long, and as the probability of having an available continuous space

of more than 30 ft. on each side of a partition fully occupied with bookcases, completely filled, is extremely small; and when it is considered that this load will not be carried entirely by one beam, the floor acting as a distributor, this may safely be neglected, and it may be assumed that a concentrated load of 5 000 lb. covers all ordinary contingencies.

In order to have a comparison between the system of concentrated loading and the uniform loads usually specified, the following table gives the equivalent loads per square foot for beams of different lengths and spacing.

Span of beam, in feet.	Distance between Centers of Beams, in Feet.			
	4	5	6	7
10	250	200	166	143
15	166	133	111	95
20	125	100	83	71
25	100	80	66	57
30	83	66	55	48
35	72	57	48	41
40	62	50	42	36

The application of a concentrated load to each beam has the additional advantage of having all beam connections proportioned for a load of 5 000 lb., and, therefore, not only obtains stronger connections and additional stiffness, but also makes provision for excessive concentrated loads during erection.

A concentrated load of 5 000 lb. applied to floor girders, that is, girders which carry beams, is not sufficient to cover all cases of extreme loading.

By laying out a great number of arrangements of office floors, with different spacings of columns and beams, and by applying different combinations of maximum loads, it was found that a uniform load of 1 000 lb. per lin. ft. of girder will generally cover all possible contingencies, unless the uniform load of 40 lb. per sq. ft. gives greater results.

The floor girders, therefore, have to be tried by three methods in order to ascertain which gives the greatest result:

First.—For a concentrated load of 5 000 lb.;

Second.—A uniform load of 1 000 lb. per lin. ft.;

Third.—A uniform load of 40 lb. per sq. ft. of floor area.

This method of loading, as specified for office buildings, will also

provide ample safety if any rooms should be used for light manufacturing purposes.

By applying, in actual examples, the rules for floor loads of office buildings, as recommended by the writer, it is found, if the columns are spaced at 20 ft. between centers in either direction, and the beams at 5 ft. between centers, that the live load on the beams will be equivalent to 100 lb., and on the girders 50 lb., per sq. ft. of floor area.

If the columns were spaced at 25 ft. between centers, and the beams 5 ft., as before, the live load would be equal to 80 lb. per sq. ft. on the beams and 40 lb. per sq. ft. on the girders.

For buildings occupied as dwellings, a concentrated load of 2 000 lb. is recommended for beams and a uniform load of 500 lb. per lin. ft. for girders, in connection with a uniform floor load of 40 lb. per sq. ft.

As it has been demonstrated that a uniform load of 40 lb. per sq. ft. will scarcely ever be exceeded by a crowd of people, this load, with the excess loads specified for office buildings, will be sufficient for floors of schools, churches, theaters and places where seats are provided, but for places where strong vibrations may be expected, such as ballrooms, drillrooms, gymnasia, etc., 100% should be added to the uniform live load for impact and vibrations, and, in order to reduce the deflections, and consequently the vibrations, the depth of the beams and girders should be limited to one-fifteenth of their span.

Loads on Columns.

It has been the practice of many engineers and architects to allow smaller live loads on columns than those specified for the floor system, or to reduce the loads per square foot of floor area on columns from story to story downward toward the foundations. Rules to that effect are also incorporated in some building laws.

In the specifications proposed by the writer, the rules of the New York building laws have been adopted, *viz.*, to reduce the live load on columns in buildings more than five stories high 5% for each story (commencing with the columns carrying the second floor from the top), until a reduction of 50% is reached.

In order to provide for any possible excessive loads, and to keep the dimensions of the lighter columns within reasonable limits, it is specified that columns carrying floor loads shall be proportioned for a minimum live load of 20 000 lb., and the proportion of length divided by least radius of gyration of section shall be limited to 125.

Applying these rules to the columns of an office building gives, for the columns carrying the top floor, a live load of 40 lb. per sq. ft. of floor area (unless this load is exceeded by the concentrated load of 20 000 lb.), which is reduced to a minimum load of 20 lb. per sq. ft. This agrees very closely with the investigations of Messrs. Blackall and Everett, who found the average maximum live load on any one office floor to be 40.2 lb. per sq. ft., and the average total maximum for any one building to be 17 lb. per sq. ft.

Loads on Foundations.

Several failures, which have resulted from unequal settlement of foundations, have demonstrated that it is of the utmost importance that foundations should be proportioned properly, more particularly those not on solid rock, or where a settlement is to be expected. The reason for some of these failures was that the foundations were proportioned for the theoretical live loads, which never occurred. During construction, when the dead loads caused a settlement, those foundations which received the smallest amount of live load, therefore, were strained nearly to the limit of the permissible pressure, while those with a large amount of live load received a very much smaller pressure per square foot, thus causing unequal settlement.

For many years it has been the practice to reduce the live loads on foundations to less than the amounts allowed on the footing of columns, a provision which also exists in some building laws. However, as the average live load in a fire-proof building is only 20 lb. (or less) per sq. ft., and the dead load on the interior columns approximately 100 lb per sq. ft., and on exterior columns considerably more; and, as the foundations have generally reached their maximum settlement before the building is occupied, it seems that the logically correct way to proportion foundations would be to consider the dead load only, but reduce the pressure per square foot so that the permissible pressure for combined dead and live load will not be exceeded. For example:

The base of that foundation which gets the greatest live load in proportion to the dead load has:

100 000 lb.	live load,
400 000 "	dead load,
500 000 "	total load.

The permissible pressure on the base of the foundation is 2 tons, or 4 000 lb. per sq. ft., and this foundation, therefore, would require an area of $\frac{500\,000}{4\,000} = 125$ sq. ft., which, if the live load be omitted, would give a pressure of 3 200 lb. per sq. ft. for dead load alone. Therefore, all foundations in this building should be proportioned so that the pressure from the dead load alone will not exceed 3 200 lb. per sq. ft.

Wind Loads.

All structures are exposed to high wind pressures occasionally, and there have been many disasters caused by structures having been blown down by the wind. All cases of this kind can be traced to inadequate provision in the design to resist these forces. It is not sufficient to compute the wind strains on the exposed surface of the finished building and depend upon the walls and partitions for bracing. The steel frame of a building is generally run up ahead of the walls and partitions. In several instances the framework of buildings has been wrecked during erection, or has been blown out of plumb and has had to be pulled back into place. Sometimes, temporary wooden braces or temporary adjustable rods have been used to hold the framework in line during erection. In one case the framework was so flimsy and shaky that the erectors were afraid to work on it, and, in order to make it safe during erection, tied it together with wire ropes. Certainly, this was not good practice. The steel frame of a building should be treated as an independent structure, the same as the towers of a viaduct, and should be able to resist the wind forces on all surfaces exposed during erection. This should be accomplished by substantial bracing, or by designing the columns and connections so that they may be able to resist the bending strains produced by wind pressure. No temporary makeshifts should be allowed. This method has the advantage of imparting additional stiffness to the framework.

In proportioning the members of the structure for these temporary wind strains, it is permissible to allow a higher unit strain than for permanent work, say 20 000 lb. per sq. in., or about two-thirds of the elastic limit.

Unit Strains.

The permissible pressure allowed on foundations on different kinds of soil, on concrete, masonry, brickwork, etc., have been compiled from different sources.

The permissible unit strains on steel are specified as 16 000 lb. per sq. in., which is approximately one-half of the elastic limit; therefore, giving a factor of safety of 2. This is in accordance with the best practice now in vogue for bridge work.

Material and Workmanship.

It was not deemed necessary to include in these specifications the quality of the ordinary building material, such as cement, concrete, stone masonry, brickwork, etc., as most of that material is of a local character and is generally well covered by architects' and engineers' specifications.

Cast Iron.—This is practically ruled out in these specifications, as it is the poorest of all metals used for structural purposes to resist bending and tension. It has been the cause of several disasters, and, in bridge work, has been entirely abandoned for many years. The use of cast iron in columns with the usual beam connections is to be particularly condemned, as the beams are supported by lugs or brackets cast on the columns, thus producing eccentric loading and bending strains.

Rolled Steel.—This material, of the grade called "structural steel," adopted by the American Railway Engineering and Maintenance-of-Way Association for bridge material, is specified for all structural parts, as it is considered the most reliable for structural purposes. It is, moreover, a commercial article which can be purchased from any reputable manufacturer without extra cost.

The specifications for material and workmanship are practically the same as those adopted by the American Railway Engineering and Maintenance-of-Way Association, as far as they were applicable to structural work for buildings.

These specifications are divided into two parts:

Part I.—This contains the information necessary for computation and designing, such as loads, unit strains and details of construction.

Part II.—This covers the quality of material, the workmanship and the inspection.

This division is made so that each part may be used separately: Part I in the office, by the designer, and Part II in the shop, by the manufacturer and inspector, and, for this reason, the paragraphs in each part are numbered separately.

GENERAL SPECIFICATIONS FOR STRUCTURAL STEELWORK OF BUILDINGS.

PART I.—DESIGN.

Loads.

1.—Dead Load.—The "dead" load in all structures shall consist of the weight of walls, floors, partitions, roofs and all other permanent construction and fixtures.

2.—In calculating the "dead" loads, the weights of the different materials shall be assumed as given in Table 1.

3.—Live Load on Floors.—The following table gives the "live" load on floors, to be assumed for different classes of buildings. These loads consist of:

a.—A uniform load per square foot of floor area;

b.—A concentrated load which shall be applied to all points of the floor;

c.—A uniform load per linear foot for girders.

The maximum result is to be used in calculations.

The specified concentrated loads shall also apply to the floor construction between the beams for a length of 5 ft.

Live Loads.

Classes of buildings.	Live Loads, in Pounds. Distributed load.	Concentrated load.	Load per linear foot of girder.
Dwellings, hotels and apartment-houses..	40	2 000	500
Office buildings..............................	40	5 000	1 000
Assembly rooms with fixed seats, like theaters, churches, schools, etc..........	40	5 000	1 000
Assembly rooms, without fixed seats, like ballrooms, gymnasia, armories, etc.....	80	5 000	1 000
Stables and carriage-houses..............	70	5 000	1 000
Ordinary stores and light manufacturing.	40	8 000	1 000
Sidewalks in front of buildings............	100	10 000	......
Warehouses and factories..................	from 120 up	Special.	Special.
Charging floors for foundries..............	" 300 "	"	"
Power-houses, for uncovered floors........	" 200 "	The actual weights of engines, boilers, stacks, etc., shall be used, but in no case less than 200 lb. per sq. ft.	

4.—If heavy concentrations, like safes, armatures, or special machinery, are likely to occur on floors, provision should be made for them.

5.—Crane Loads and Impact.—For structures carrying traveling machinery, such as cranes, conveyors, etc., 25% shall be added to the strains resulting from such live load, to provide for the effects of impact and vibrations. (For crane loads, see Table 2.)

6.—Loads on Flat Roofs.—Flat roofs of office buildings, hotels, apartment-houses, etc., which are likely to be loaded by crowds of people, shall be treated as floors, and the same live loads shall be used as specified for hotels and dwelling-houses.

7.—Loads on Ordinary Roofs.—Ordinary roofs shall be proportioned to carry the following loads per square foot of exposed surface, applied vertically, to provide for dead and live loads combined:

Gravel or composition roofing.	On boards, flat pitch, 3 to 12 in., or less	45 lb.
	On boards, steep pitch, more than 3 to 12 in.	40 "
	On 3-in. flat tile or cinder concrete	55 "
Corrugated sheeting, on boards or purlins		40 "
Slate.	On boards or purlins	50 "
	On 3-in. flat tile or cinder concrete	65 "
Tile on steel purlins		55 "

For roofs in climates where no snow is likely to occur, reduce the foregoing total loads by 10 lb. per sq. ft.

8.—Large Roofs.—Large roofs, such as train-sheds, armories, public halls, etc., shall be proportioned to carry, in addition to their own weight:

a.—A live load, representing snow, per horizontal square foot of roof of:

15 lb. for all slopes not exceeding 35°;

10 lb. for all slopes between 35 and 45 degrees.

The possibility of a partial snow loading has to be considered. The snow load can be neglected in certain climates, also in roofs having slopes exceeding 45°, if there are no snow guards or other obstructions.

b.—Wind loads as specified in Paragraph 12.

9.—Loads on Columns.—For columns, the specified uniform live loads per square foot shall be used, with a minimum of 20 000 lb. per column.

10.—Reduction of Live Load on Columns.—For buildings more than five stories in height, these live loads may be reduced as follows:

For roof and top floor, no reduction;

For each succeeding lower floor, a reduction of 5% until 50% is reached, which is to be used for the columns of all remaining floors, *viz.*, the reduced load is to be used for the total floor area carried by the column.

11.—Loads on Foundations.—The live loads on foundations shall be assumed to be the same as for the footings of columns. The areas of the bases of the foundations shall be proportioned for the dead load only. That foundation which receives the largest ratio of live to dead load shall be selected and proportioned for the combined dead and live loads. The dead load on this foundation shall be divided by the area thus found, and this reduced pressure per square foot shall be the permissible working pressure to be used for the dead load of all foundations.

12.—Wind Pressure.—The wind pressure shall be assumed at 30 lb. per sq. ft. acting in either direction horizontally:

First.—On the sides and ends of buildings and on the actually exposed surface, or the vertical projection of roofs;

Second.—On the total exposed surfaces of all parts composing the metal framework. The framework shall be considered an independent structure, without walls, partitions or floors.

Unit Strains.

Substructure.

13.—Foundations.—Permissible pressure on foundations, in tons per square foot:

Soft clay, and wet sand	1
Ordinary clay, and dry sand mixed with clay	2
Dry sand, and dry clay	3
Hard clay, and firm, coarse sand	4
Firm, coarse sand and gravel	6

14.—Masonry.—Permissible working pressure in masonry, in tons per square foot:

Common brick, lime mortar	7
" " Rosendale cement mortar	8
" " Portland cement mortar	10
Hard-burned brick, Portland cement mortar	12
Rubble masonry, lime mortar	5
" " Rosendale cement mortar	6
" " Portland cement mortar	8
Coursed rubble, Portland cement mortar	10

Concrete for walls:

Rosendale cement,		1-2-5	8
"	"	1-2-4	9
Portland	"	1-2-5	15
"	"	1-2-4	16

15.—Pressure on Wall-Plates.—The pressure of beams, girders, wall-plates, column bases, etc., on masonry shall not exceed the following, in pounds per square inch:

On brickwork with cement mortar	150
" rubble masonry with cement mortar	150
" Portland cement concrete	250
" first-class masonry, sandstone	200 to 300
" " " " limestone	300 to 500
" " " " granite	400 to 800

16.—Bearing Power of Piles.—The maximum load carried by any pile shall not exceed 40 000 lb. Piles driven in loose, wet soil shall not be strained to more than 350 lb. per sq. in. of their average cross-section.

The safe load on wooden piles shall be determined by the following formula:*

$$P = \frac{2\ W\ H}{s+1}.$$

Where P = safe load on pile, in tons;

H = distance of free fall of hammer, in feet;

W = weight of hammer, in tons;

s = penetration of the pile for the last blow, in inches.

Superstructure.

Steel.

17.—Permissible Strains.—All parts of the structure shall be proportioned so that the sum of the dead and live loads, together with the impact, if any, shall not cause the strains to exceed those given in the following table:

	Pounds per square inch.
Tension, net section	16 000
Direct compression	16 000
Shear, on rivets and pins	12 000
Shear, on bolts	8 000
Shear, on plate-girder web (gross section)	10 000
Bearing pressure, on pins and rivets	24 000
Bearing pressure, on bolts	16 000
Fiber strain, on pins	24 000

**Engineering News* formula.

18.—For wind bracing, and the combined strains due to wind and the other loading, the permissible working strains may be increased 25%, or to 20 000 lb. for direct compression or tension.

19.—Permissible Compression Strains.—For compression members, the permissible strains of 16 000 and 20 000 lb. per sq. in. shall be reduced by the following formula:

$$p = 16\,000 - 70\,\frac{l}{r}$$

$$p = 20\,000 - 90\,\frac{l}{r}$$

Where p = permissible working strain per square inch in compression;

l = length of piece, in inches, from center to center of connections;

r = least radius of gyration of the section, in inches.

20.—Provision for Eccentric Loading.—In proportioning columns, provision must be made for eccentric loading.

21.—Expansion Rollers.—The pressure per linear inch on expansion rollers shall not exceed 600 d, where d = diameter of rollers, in inches.

22.—Transverse Loading of Tension or Compression Members.—When a floor, wall or other weight rests directly on the chord of a truss, said chord shall be proportioned so that the sum of the strains per square inch on the outer fiber, resulting from direct compression or tension, and three-fourths of the maximum bending moment (the chord being considered as a beam of one panel length, supported at the ends) shall not exceed the specified limiting strains in tension or compression, the proper amount of impact, if any, being added to each kind of loading.

23.—The bending moments at panel points shall be assumed equal to that in the center, but in opposite direction.

24.—Combined Strains.—All other members which are subject to direct strain, in addition to bending moment, shall be calculated in a similar manner.

25.—Alternate Strains.—Members and connections subject to alternate strains shall be proportioned for the strain giving the largest section.

26.—Net Sections.—Net sections must be used in calculating tension members, and, in deducting rivet holes, they must be taken ⅛ in. larger than the nominal size of the rivets.

27.—Pin-connected riveted tension members shall have a net section through the pin holes 25% in excess of the net section of the body of the member. The net section back of the pin hole shall be at least 0.75 of the net section through the pin hole.

28.—Compression Members Limiting Length.—No compression member shall have a length exceeding 125 times its least radius of gyration, except those for wind and lateral bracing, which may have a length not exceeding 150 times the least radius of gyration.

29.—Plate Girders.—Plate girders shall be proportioned on the assumption that one-eighth of the gross area of the web is available as flange area. The compression flange shall have the same sectional area as the tension flange, but the unsupported length of the flange shall not exceed 30 times its width.

30.—In plate girders used as crane runways, the unsupported length of the compression flange shall not exceed 20 times its width.

31.—Web Stiffeners.—The web shall have stiffeners at the ends and inner edges of bearing plates, and at all points of concentrated loads, and also at intermediate points, when the thickness of the web is less than one-sixtieth of the unsupported distance between flange angles, generally not farther apart than the depth of the full web plate, with a minimum limit of 5 ft.

32.—Rolled Beams.—I-beams, and channels used as beams or girders, shall be proportioned by their moments of inertia.

33.—Limiting Depth of Beams and Girders.—The depth of rolled beams in floors shall be not less than one-twentieth of the span, and if used as roof purlins, not less than one-thirtieth of the span.

In case of floors subject to shocks and vibrations, the depth of beams and girders shall be limited to one-fifteenth of the span.

34.—Field Connections.—In field connections, the number of rivets shall be increased 15 per cent.

Cast Iron.

35.—Compression on Cast Iron.—The direct compression on cast iron shall not exceed 12 000 lb. per sq. in.

Timber.

36.—Timber.—The timber parts of the structure shall be proportioned in accordance with the following unit strains, given in pounds per square inch.

Kind of timber.	Transverse loading.	End bearing.	Columns under 12 diameters.	Bearing across fiber.	Shear along fiber.
White Oak	1 300	1 300	1 000	500	200
Long-Leaf Pine	1 500	1 500	1 000	350	100
White Pine and Spruce	1 000	1 000	700	200	100
Hemlock	800	800	650	200	100

37.—Columns, the length of which exceeds twelve times their least diameter, shall be proportioned by the following formula:

$$p = \frac{C}{1 + \frac{l^2}{1\,000\,d^2}}$$

Where C = unit strains, as given in Paragraph 36 for short columns;

l = length of column, in inches;

d = least side of column, in inches.

38.—Planking.—For thickness of floor and roof planking, see Table 3.

Details of Construction.

39.—Minimum Thickness of Material.—No steel less than ¼ in. thick shall be used, except for lining or filling vacant spaces.

40.—Adjustable Members.—Adjustable members in any part of structures shall preferably be avoided.

41.—Symmetrical Sections.—Sections shall preferably be made symmetrical.

42.—Connections.—The strength of connections shall be sufficient to develop the full strength of the member.

43.—No connection, except lattice bars, shall have less than two rivets.

44.—Floor Beams.—Floor beams shall generally be rolled steel beams, and the ends shall be attached to the webs of the floor girders with angle connections.

45.—For fire-proof floors, they shall be arranged, as to spacing and length, so that the dead and live loads together shall not cause a greater deflection of the beams than $\frac{1}{30}$ in. per foot of span. They shall generally be tied together with tie-rods at intervals not exceeding eight times the depth of the beams. Holes for tie-rods, where the

construction of the floor permits, shall be spaced about 3 in. above the bottom of the beam.

46.—Beam Girder.—When more than one rolled beam is used to form a girder, they shall be connected by bolts and separators at intervals of not more than 5 ft. All beams having a depth of 12 in. and more shall have at least two bolts to each separator.

47.—Wall Ends of Beams and Girders.—Wall ends of a sufficient number of joists and girders shall be anchored securely, to impart rigidity to the structure.

48.—Wall-Plates and Column Bases.—Wall-plates and column bases shall be constructed so that the load will be well distributed over the entire bearings. If they do not get the full bearing on the masonry, the deficiency shall be made good with rust cement or Portland cement mortar.

49.—Floor Girders.—The floor girders may be rolled beams or plate girders; they shall preferably be riveted or bolted to columns by means of connection angles. Shelf-angles or other supports may be provided for convenience during erection.

50.—Flange Plates.—The flange plates of all girders shall be limited in width, so as not to extend, beyond the outer line of rivets connecting them to the angles, more than 6 in., or more than eight times the thickness of the thinnest plates.

51.—Web Stiffeners.—Web stiffeners shall be in pairs, and shall have a close bearing against the flange angles. Those over the end bearing, or forming the connection between girder and column, shall be on fillers. Intermediate stiffeners may be on fillers or crimped over the flange angles. The rivet pitch in stiffeners shall not be more than 5 in.

52.—Web Splices.—Web plates of girders must be spliced at all points by a plate on each side of the web, capable of transmitting the full strain through splice rivets.

53.—Columns.—Columns shall be designed so as to provide for effective connections of floor beams, girders or brackets.

They shall preferably be continuous over several stories.

54.—Column Splices.—The splices shall be strong enough to resist the bending strain and make the columns practically continuous for their whole length.

55.—Trusses.—Trusses shall preferably be riveted structures.

Heavy trusses, of long span, where the riveted field connection would become unwieldy, or for other good reasons, may be designed as pin-connected structures.

56.—Intersecting Members.—Main members of trusses shall be designed so that the neutral axes of intersecting members shall meet in a common point.

57.—Roof Trusses.—Roof trusses shall be braced in pairs in the plane of the chords.

Purlins shall be made of shapes, or riveted-up plate, or lattice girders.

Trussed purlins will not be allowed.

58.—Eye-Bars.—The eye-bars in pin-connected trusses composing a member shall be as nearly parallel to the axis of the truss as possible.

59.—Spacing of Rivets.—The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than 3 in. for $\frac{7}{8}$-in. rivets, $2\frac{5}{8}$ in. for $\frac{3}{4}$-in. rivets, $2\frac{1}{4}$ in. for $\frac{5}{8}$-in. rivets and $1\frac{3}{4}$ in. for $\frac{5}{8}$-in. rivets.

60.—For angles with two gauge lines, the maximum shall be twice as great as given in Paragraph 59 in each line with rivets staggered; and, where two or more plates are used in contact, rivets not more than 12 in. apart in any direction shall be used to hold the plates together.

61.—The pitch of the rivet, in the direction of the strain, shall not exceed 6 in., nor 16 times the thinnest outside plate connected, and not more than 50 times that thickness at right angles to the strain.

62.—Edge Distance.—The minimum distance from the center of any rivet hole to a sheared edge shall be $1\frac{1}{2}$ in. for $\frac{7}{8}$-in. rivets, $1\frac{1}{4}$ in. for $\frac{3}{4}$-in. rivets, $1\frac{1}{8}$ in. for $\frac{5}{8}$-in. rivets and 1 in. for $\frac{1}{2}$-in. rivets, and to the rolled edge, $1\frac{1}{4}$, $1\frac{1}{8}$, 1 and $\frac{7}{8}$ in., respectively.

63.—The maximum distance from any edge shall be eight times the thickness of the plate.

64.—Maximum Diameter.—The diameter of the rivets in any angle carrying calculated strains shall not exceed one-quarter of the width of the leg in which they are driven. In minor parts, rivets may be $\frac{1}{8}$ in. greater in diameter.

65.—Pitch at Ends.—The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets for a length equal to one and one-half times the maximum width of the member.

66.—Tie-Plates.—The open sides of compression members shall be provided with lattice having tie-plates at each end and at intermediate points where the lattice is interrupted. The tie-plates shall be as near the ends as practicable. In main members, carrying calculated strains, the end tie-plates shall have a length not less than the distance between the lines of rivets connecting them to the flanges, and intermediate ones not less than half this distance.

Their thickness shall be not less than one-sixtieth of the same distance.

67.—Lattice.—The thickness of lattice bars shall be not less than one-fortieth for single lattice and one-sixtieth for double lattice, of the distance between end rivets; their width shall be in accordance with the following:

For 15-in. channels, or built sections with $3\frac{1}{2}$ and 4-in. angles..................	$2\frac{1}{2}$ in. ($\frac{7}{8}$-in. rivets);
For 12, 10 and 9-in. channels, or built sections with 3-in. angles............	$2\frac{1}{4}$ in. ($\frac{3}{4}$-in. rivets);
For 8 and 7-in. channels, or built sections with $2\frac{1}{2}$-in. angles.............	2 in. ($\frac{5}{8}$-in. rivets);
For 6 and 5-in. channels, or built sections with 2-in. angles...............	$1\frac{3}{4}$ in. ($\frac{1}{2}$-in. rivets).

68.—Lattice bars with two rivets shall generally be used in flanges more than 5 in. wide.

69.—Angle of Lattice.—The inclination of lattice bars with the axis of the member, generally, shall be not less than 45°, and when the distance between the rivet lines in the flanges is more than 15 in., if a single rivet bar is used, the lattice shall be double and riveted at the intersection.

70.—Spacing of Lattice.—The pitch of lattice connections, along the flange divided by the radius of gyration of the flange angle about an axis, through its center of gravity, perpendicular to the plane of the lattice, shall be less than the corresponding ratio of the member as a whole.

71.—Faced Joints.—Abutting joints in compression members when faced for bearing shall be spliced sufficiently to hold the connecting members accurately in place.

72.—All other joints in riveted work, whether in tension or compression, shall be fully spliced.

73.—Pin Plates.—Pin holes shall be reinforced by plates where necessary; and at least one plate shall be as wide as the flange will allow. Where angles are used, the plates shall be on the same side as the angles; they shall contain sufficient rivets to distribute their portion of the pin pressure to the full cross-section of the member.

74.—Pins.—Pins shall be long enough to insure a full bearing of all parts connected upon the turned-down body of the pin.

75.—Members packed on pins shall be held against lateral movement.

76.—Bolts.—Where members are connected by bolts, the body of these bolts shall be long enough to extend through the metal. A washer at least $\frac{3}{16}$ in. thick shall be under the nut.

77.—Fillers.—Fillers between parts carrying strain shall have a sufficient number of independent rivets to transmit the strain to the member to which the filler is attached.

78.—Temperature.—Provision shall be made for expansion and contraction, corresponding to a variation of temperature of 150° fahr., where necessary.

79.—Rollers.—Expansion rollers shall be not less than 4 in. in diameter.

80.—Stone Bolts.—Stone bolts shall extend not less than 4 in. into granite pedestals and 8 in. into other material.

81.—Anchorage.—Columns which are strained in tension at their base shall be anchored to the foundations.

82.—Anchor bolts shall be long enough to engage a mass of masonry, the weight of which shall be one and one-half times the tensile strain in the anchor.

83.—Bracing.—Lateral, longitudinal and transverse bracing in all structures shall be preferably composed of rigid members.

PART II.—MATERIAL AND WORKMANSHIP.

Material.

1.—Steel.—All parts of the structures shall be of rolled steel, except column bases, bearing plates or minor details, which may be of cast iron or cast steel. No cast iron shall be used in pieces which will have to resist tension or bending strains.

2.—Process of Manufacture.—Steel may be made by the open-hearth or by the Bessemer process.

3.—The chemical and physical properties shall conform to the following limits.

Schedule of Requirements of Steel.

Chemical and physical properties.	Structural steel.	Rivet steel.	Steel castings.
Phosphorus, maximum.. Basic..	0.04 per cent.	0.04 per cent.	0.05 per cent.
Phosphorus, maximum.. Acid...	0.08 per cent.	0.04 per cent.	0.08 per cent.
Sulphur, maximum...............	0.05 per cent.	0.04 per cent.	0.05 per cent.
Ultimate tensile strength, Pounds per square inch..........	Desired 60 000	Desired 50 000	Not less than 65 000
Elongation: minimum percentage in 8 in....................	1 500 000* / Ult. tensile str'gth	1 500 000 / Ult. tensile str'gth	....
Elongation: minimum percentage in 2 in....................	22	....	18
Character of fracture............	Silky	Silky	Silky or fine granular.
Cold bends without fracture.....	180° flat.†	180° flat§	90°

* See Paragraph 12. † See Paragraphs 14 and 15. § See Paragraph 16.

4.—The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

5.—Allowable Variations.—Tensile tests of steel showing an ultimate strength within 5 000 lb. of that desired will be considered satisfactory.

6.—Chemical Analyses.—Chemical determinations of the percentages of carbon, phosphorus, sulphur and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector.

7.—Form of Specimens for Plates, Shapes and Bars.—Specimens for tensile and bending tests, for plates, shapes and bars, shall be made by cutting coupons from the finished product, which shall have both

faces rolled and both edges milled to the form shown by Fig. 1; or with both edges parallel; or they may be turned to a diameter of ¾ in. for a length of at least 9 in., with enlarged ends.

8.—Rivets.—Rivet rods shall be tested as rolled.

Pins and Rollers.—Specimens shall be cut from the finished rolled or forged bar, in such manner that the center of the specimen shall be 1 in. from the surface of the bar. The specimen for the tensile test shall be turned to the form shown by Fig. 2. The specimen for the bending test shall be 1 in. by ½ in. in section.

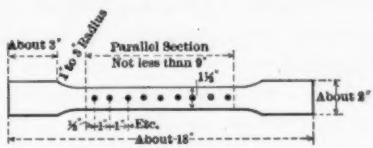


FIG. 1.

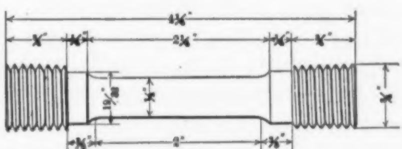


FIG. 2.

9.—Steel Castings.—The number of tests will depend on the character and importance of the castings. Specimens shall be cut cold from coupons moulded and cast on some portion of one or more castings from each melt, or from the sink-heads, if the heads are of sufficient size. The coupon or sink-head, so used, shall be annealed with the casting before it is cut off. Test specimens shall be of the form prescribed for pins and rollers.

10.—Annealed Specimens.—Material which is to be used without annealing or further treatment shall be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen for tensile tests representing such material shall be cut from properly annealed or similarly treated short lengths of the full section of the bar.

11.—Number of Tests.—At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing ⅜ in. and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

12.—Modifications in Elongation.—For material less than $\frac{5}{16}$ in. and more than $\frac{3}{4}$ in. in thickness, the following modifications will be allowed in the requirements for elongation:

a.—For each $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in., a deduction of $2\frac{1}{2}\%$ will be allowed from the specified elongation.

b.—For each $\frac{1}{8}$ in. in thickness above $\frac{3}{4}$ in., a deduction of 1% will be allowed from the specified elongation.

c.—For pins and rollers more than 3 in. in diameter, the elongation in 8 in. may be 5% less than that specified in Paragraph 3.

13.—Bending Tests.—Bending tests may be made by pressure or by blows. Plates, shapes and bars less than 1 in. thick shall bend as called for in Paragraph 3.

14.—Thick Material.—Full-sized material, for eye-bars and other steel 1 in. or more in thickness, tested or rolled, shall bend cold 180° around a pin the diameter of which is equal to twice the thickness of the bar, without fracture on the outside of the bend.

15.—Bending Angles.—Angles $\frac{3}{4}$ in. and less in thickness shall open flat, and angles $\frac{1}{2}$ in. and less in thickness shall bend shut, cold, under blows of a hammer, without sign of fracture. This test will be made only when required by the inspector.

16.—Nicked Bends.—Rivet steel, when nicked and bent around a bar of the same diameter as the rivet rod, shall give a gradual break and a fine, silky uniform fracture.

17.—Finish.—Finished material shall be free from injurious seams, flaws, cracks, defective edges, or other defects, and shall have a smooth, uniform, workmanlike finish. Plates 36 in. and less in width shall have rolled edges.

18.—Stamping.—Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it. Steel for pins and rollers shall be stamped on the end. Rivet and lattice steel and other small parts may be bundled with the above marks on an attached tag.

19.—Defective Material.—Material which, subsequent to the foregoing tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious defects, will be rejected at the shop, and shall be replaced by the manufacturer at his own cost.

20.—Allowable Variation in Weight.—A variation in cross-section or

weight of each piece of steel of more than $2\frac{1}{2}\%$ from that specified will be sufficient cause for rejection.

21.—Cast Iron.—Iron castings shall be made of tough, gray iron, free from injurious cold-shuts or blow-holes, true to pattern, and of workmanlike finish. Test pieces 1 in. square shall be capable of sustaining on a clear span of 12 in. a central load of 2 500 lb. or more, and deflect at least 0.15 in. before rupture.

Workmanship.

22.—General.—All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works.

23.—Straightening Material.—Material shall be thoroughly straightened in the shop, by methods which will not injure it, before being laid off or worked in any way.

24.—Finish.—Shearing shall be done neatly and accurately, and all portions of the work exposed to view shall be finished neatly.

25.—Rivets.—The size of rivets called for on the plans shall be understood to mean the actual size of the cold rivet before heating.

26.—Rivet Holes.—The diameter of the punch for material not more than $\frac{5}{8}$ in. thick shall be not more than $\frac{1}{16}$ in., nor that of the die more than $\frac{1}{8}$ in., larger than the diameter of the rivet. Material more than $\frac{5}{8}$ in. thick, except minor details, shall be sub-punched and reamed or drilled from the solid.

27.—Punching.—Punching shall be done accurately. Slight inaccuracy in the matching of holes may be corrected with reamers. Drifting to enlarge unfair holes will not be allowed. Poor matching of holes will be cause for rejection, at the option of the inspector.

28.—Assembling.—Riveted members shall have all parts well pinned up and firmly drawn together with bolts before riveting is commenced. Contact surfaces shall be painted. (See Paragraph 52.)

29.—Lattice Bars.—Lattice bars shall have neatly rounded ends, unless otherwise called for.

30.—Web Stiffeners.—Stiffeners shall fit neatly between the flanges of girders. Where tight fits are called for, the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

31.—Splice Plates and Fillers.—Web splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$ in. of flange angles.

32.—Connection Angles.—Connection angles for floor girders shall be flush with each other, and correct as to position and length of girder.

33.—Riveting.—Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

34.—Rivets shall look neat and finished, with heads of approved shape, full, and of equal size. They shall be central on the shank and shall grip the assembled pieces firmly. Recupping and caulking will not be allowed. Loose, burned, or otherwise defective rivets shall be cut out and replaced. In cutting out rivets, great care shall be taken not to injure the adjoining metal. If necessary, they shall be drilled out.

35.—Bolts.—Wherever bolts are used in place of rivets which transmit shear, such bolts must have a driving fit. A washer not less than $\frac{1}{4}$ in. thick shall be used under the nut.

36.—Members to be Straight.—The several pieces forming one built member shall be straight and shall fit closely together, and finished members shall be free from twists, bends or open joints.

37.—Finish of Joints.—Abutting joints shall be cut or dressed true and straight and fitted close together, especially where open to view In compression joints depending on contact bearing, the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

38.—Eye-Bars.—Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of the heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to break in the body with a silky fracture, when tested to rupture. The thickness of the head and neck shall not vary more than $\frac{1}{16}$ in. from the thickness of the bar.

39.—Boring Eye-Bars.—Before boring, each eye-bar shall be perfectly annealed and carefully straightened. Pin holes shall be in the center line of bars and in the center of heads. Bars of the same length shall be bored so accurately that, when placed together, pins $\frac{1}{32}$ in. smaller in diameter than the pin holes can be passed through the holes at both ends of the bars at the same time.

40.—Pin Holes.—Pin holes shall be bored true to gauges, smooth and straight; at right angles to the axis of the member, and parallel to each other, unless otherwise called for. Wherever possible, the boring shall be done after the member is riveted up.

41.—The distance from center to center of pin holes shall be correct within $\frac{1}{32}$ in., and the diameter of the hole not more than $\frac{1}{50}$ in. larger than that of the pin, for pins up to 5 in. diameter, and $\frac{1}{32}$ in. for larger pins.

42.—Pins and Rollers.—Pins and rollers shall be turned accurately to gauges, and shall be straight, smooth and entirely free from flaws.

43.—Pilot Nuts.—At least one pilot and driving nut shall be furnished for each size of pin for each structure.

44.—Screw Threads.—Screw threads shall make tight fits in the nuts, and shall be United States standard, except at the ends of pins and for bolts more than $1\frac{1}{2}$ in. in diameter, for which six threads per inch shall be used.

45.—Annealing.—Steel, except in minor details, which has been partially heated shall be properly annealed.

46.—Steel Castings.—All steel castings shall be annealed.

47.—Welds.—Welds in steel will not be allowed,

48.—Bed-Plates.—Expansion bed-plates shall be planed true and smooth. Cast wall-plates shall be planed at top and bottom. The cut of the planing tool shall correspond with the direction of expansion.

49.—Shipping Details.—Pins, nuts, bolts, rivets and other small details shall be boxed or crated.

50.—Weight.—The weight of every piece and box shall be marked on it in plain figures.

Painting.

51.—Shop Painting.—Steelwork, before leaving the shop, shall be thoroughly cleaned and given one good coating of pure linseed oil, or such paint as may be called for, well worked into all joints and open spaces.

52.—In riveted work, the surfaces coming in contact shall be painted before being riveted together.

53.—Pieces and parts which are not accessible for painting after erection, shall have two coats of paint before leaving the shop.

54.—Steelwork to be embedded in concrete shall not be painted.

55.—Painting shall be done only when the surface of the metal is perfectly dry. It shall not be done in wet or freezing weather, unless protected under cover.

56.—Machine-finished surfaces shall be coated with white lead and tallow before shipment, or before being put out into the open air.

57.—*Field Painting.*—After the structure is erected, the metalwork shall be painted thoroughly and evenly with an additional coat of paint, mixed with pure linseed oil, of such quality and color as may be selected.

Inspection and Testing.

58.—The manufacturer shall furnish all facilities for inspecting and testing the weight, quality of material and workmanship. He shall furnish a suitable testing machine for testing the specimens, as well as prepare the pieces for the machine, free of cost.

59.—When an inspector is furnished by the purchaser, he shall have full access at all times to all parts of the works where material under his inspection is manufactured.

60.—The purchaser shall be furnished with complete copies of mill orders, and no material shall be rolled and no work done before he has been notified as to where the orders have been placed, so that he may arrange for the inspection.

61.—The purchaser shall also be furnished with complete shop plans, and must be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect the material and workmanship.

62.—Complete copies of shipping invoices shall be furnished to the purchaser with each shipment.

63.—If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the purchaser.

Full-Sized Tests.

64.—Full-sized parts of the structure may be tested, at the option of the purchaser. If tested to destruction, such material shall be paid for at cost, less its scrap value, if it proves satisfactory.

65.—If it does not stand the specified tests, it will be considered rejected material, and be solely at the cost of the contractor, unless he is not responsible for the design of the work.

66.—In eye-bar tests, the ultimate strength, the elastic limit and the elongation in 10 ft., unless a different length is called for, shall be recorded.

67.—In transverse tests, the lateral and vertical deflections shall be recorded.

TABLE 1.—Weights of Building Materials, etc., in Pounds per Cubic Foot.

Material.	Weight.
Paving brick	150
Common building brick	120
Soft building brick	100
Granite	170
Marble	170
Limestone	160
Sandstone	145
Slag	40
Gravel	120
Slate	175
Sand, clay and earth (dry)	100
Sand, clay and earth (wet)	120
Mortar	100
Stone concrete	130-150
Cinder concrete	70
Paving asphaltum	100
Plaster of paris	140
Glass	160
Snow, freshly fallen	10
Snow, wet	50
Spruce	25
Hemlock	25
White pine	25
Douglas fir	30
Yellow pine	40
White oak	50
Common brickwork	100-120
Rubble masonry	130-150
Ashlar masonry	140-160
Cast iron	450
Wrought iron	480
Steel	490
Plaster, ceiling	10 to 15 lb. per sq. ft.

TABLE 2.—Typical Electric Traveling Cranes.

Capacity, in tons.	Span.	Wheel base.	Maximum wheel load, in pounds.	s.	v.	Weight of Rail for: Plate girders.	Weight of Rail for: Beams.
5	40	8 ft. 6 in.	12 000	10 in.	7 ft.	40 lb. per yd.	40
	60	9 " 0 "	13 000	"	"	40 "	40
10	40	9 " 0 "	19 000	"	"	45 "	40
	60	9 " 6 "	21 000	"	"	45 "	40
15	40	9 " 6 "	26 000	"	"	50 "	50
	60	10 " 0 "	29 000	"	"	50 "	50
20	40	10 " 0 "	33 000	12 in.	8 ft.	55 "	50
	60	10 " 6 "	36 000	"	"	55 "	50
25	40	10 " 0 "	40 000	"	"	60 "	50
	60	10 " 6 "	44 000	"	"	60 "	50
30	40	10 " 6 "	48 000	"	"	70 "	60
	60	11 " 0 "	52 000	"	"	70 "	60
40	40	11 " 0 "	64 000	14 in.	9 ft.	80 "	60
	60	12 " 0 "	70 000	"	"	80 "	60
50	40	11 " 0 "	72 000	"	"	100 "	60
	60	12 " 0 "	80 000	"	"	100 "	60

1.—Wheel-load can be assumed as distributed in top flange, over a distance equal to depth of girder, with a maximum limit of 30 in.

2.—In addition to the vertical load, the top flanges of the girder shall withstand a lateral loading of two-tenths of the lifting capacity of the crane, equally divided between the four wheels of the crane.

s = side clearance from center of rail.

v = vertical " " top " "

3.—The top flanges of the crane girders shall not be of a smaller width than one-twentieth of their unsupported length.

TABLE 3.—Thickness of Spruce and White Pine Plank for Floors.

Span, in feet.	Thickness, in Inches, for Various Loads per Square Foot of Plank. lb. 30	lb. 40	lb. 50	lb. 75	lb. 100	lb. 125	lb. 150	lb. 175	lb. 200	lb. 225	lb. 250	lb. 275	lb. 300	lb. 325	lb. 350	lb. 375	lb. 400
4	0.9	1.1	1.2	1.5	1.7	1.9	2.1	2.2	2.4	2.5	2.7	2.8	2.9	3.1	3.2	3.3	3.4
5	1.2	1.4	1.5	1.9	2.1	2.4	2.6	2.8	3.0	3.2	3.4	3.5	3.7	3.8	4.0	4.1	4.3
6	1.4	1.6	1.8	2.2	2.6	2.9	3.1	3.4	3.6	3.8	4.0	4.2	4.4	4.6	4.8	4.9	5.1
7	1.7	1.9	2.1	2.6	3.0	3.3	3.7	3.9	4.2	4.5	4.7	4.9	5.2	5.4	5.6	5.8	6.0
8	1.9	2.2	2.4	3.0	3.4	3.8	4.4	4.5	4.8	5.1	5.4	5.7	5.9	6.1	....	....	....
9	2.1	2.5	2.7	3.4	3.9	4.3	4.7	5.1	5.4	5.8	6.1	....	....	....	....	....	....
10	2.4	2.7	3.1	3.7	4.3	4.8	5.2	5.6	6.0	....	....	....	....	....	....	....	....
11	2.6	3.0	3.4	4.1	4.7	5.3	5.8	....	....	....	....	....	....	....	....	....	....
12	2.9	3.3	3.7	4.5	5.2	....	....	....	....	....	....	....	....	....	....	....	....
13	3.1	3.6	4.0	4.9	5.6	....	....	....	....	....	....	....	....	....	....	....	....
14	3.4	3.9	4.3	5.3	6.1	....	....	....	....	....	....	....	....	....	....	....	....

For yellow pine use nine-tenths of the above thicknesses.

APPENDIX.

EXTRACTS FROM THE FOLLOWING BUILDING LAWS.

City.	Year.
Buffalo	1896
St. Louis	1897
Philadelphia	1899
New York	1899
Chicago	1900
Boston	1900
Minneapolis	1900
Milwaukee	1901
Baltimore	1901
District of Columbia	1902

In all the following tables:

S = Stress, in pounds per square inch;

L = Length, in inches (except in the table on page 412, where L = length in feet);

D = External diameter, or least side of rectangle, in inches;

R = Least radius of gyration of column section.

MINIMUM LIVE LOADS FOR FLOORS AND ROOFS.

Structure.	* POUNDS PER SQUARE FOOT.									
	New York.	Chicago.	Philadelphia.	Boston.	Buffalo.	Minneapolis.	Milwaukee.	District of Columbia.	Baltimore.	St. Louis.
Dwellings—one or two families.	60	40	70	50	40	50	40	50	75	70
Lodging houses, apartment houses, tenement houses, hotels, etc.	60	40	70	50	70	50	40	50	...	70
Halls, dining-rooms, cafés, offices, etc., in hotels and apartment-houses	...	...	...	...	...	...	...	75	...	...
Office buildings, first floor	150	100	100	100	70	...	60	75	...	150
Office buildings, above first floor	75	100	100	100	70	...	60	75	...	70
Halls and lobbies in office buildings	...	...	...	...	...	...	...	110	...	...
Public assembly rooms: churches, theaters, etc.	90	100	120	150	100	100	80	110	150	120
Schools	75	100	120	150	100	100	50	75	150	120
Machine shops, armories, drill-rooms, etc.	...	...	...	250	...	250	250	...	...	...
Light manufacturing and retail stores and storehouses	120	100	120	...	120	100	100	110	150	150
Heavy storehouses, warehouses, livery stables, etc.	150	100	150	250	150	...	...	200	200	150
Stairways	...	...	...	...	...	...	100*	...	...	...
Sidewalks	300	...	...	...	...	350	...	...	...	...
Roofs, per square foot of superficial surface	50†	...	30	25§	...	50	30	25	30	50
Roofs, per square foot of horizontal projection	30‡	25	...	...	40	...	...	...	...	....

Wind Loads.

	New York.	Chicago.	Philadelphia.	Boston.	Buffalo.	Minneapolis.	Milwaukee.	District of Columbia.	Baltimore.	St. Louis.
Per square foot of elevation	30	30	30	30	30	...	30	30	...	30
			25 lb. at tenth story. 2½ lb. more for each story above. 2½ lb. less " " " below. 35 lb. maximum.				30 lb. at twelfth story. 2½ lb. less at each lower story.			

* Lower supports to carry two-thirds of total weight.
† Pitch less than 20 degrees.
‡ Pitch more than 20 degrees. § Also a horizontal wind pressure of 30 lb. per sq. ft.

Permissible Reduction of Live Loads.

Live Loads under Foundations in Buildings More than Three Stories High.

Structure.		New York.	Chicago.	District of Columbia.	St. Louis.
Warehouses and factories		100%	For many-storied buildings use actual average loads, and not theoretical or occasional loads.	100%	For office buildings: a live load of 10 lb. per sq. ft. on all floors. For mercantile buildings: a live load of 20 lb. per sq. ft. on all floors.
Stores and buildings for light manufacturing purposes		75 "		75 "	
Churches, schoolhouses and places of public assembly		75 "		90 "	
Office buildings, hotels, dwellings, apartment houses, tenement houses, stables, etc.	of masonry construction	60 "		60 "	
do. do.	of steel skeleton construction	60 "		75 "	

Live Loads on Columns.

New York.		St. Louis.	
In dwellings, office buildings, stores, stables and public buildings more than five stories in height.	For roof and top-floor columns full live load. For each succeeding lower floor columns a reduction of 5% until 50% of live load remains, which is minimum.	In office and mercantile buildings.	Attic columns: full live load. Basement columns: 80% of live load and proportionately for intermediate floors.

Live Loads on Girders (St. Louis only).

	80% of full live load. (Does not apply to beams.)

Philadelphia Law: For all tenant houses, hotels, apartment houses, hospitals and office buildings, the live loads on columns, girders and foundations, may be estimated by the formula, $X = 100 - \frac{3}{8}\sqrt{A}$, and for light manufacturing buildings by the formula, $X = 100 - \frac{3}{8}\sqrt{A}$, in which X = the percentage of live load to be used, and A = area carried by any girder, column or foundation.

For permissible loads in Milwaukee Law, see the next following table.

From Milwaukee Building Law.

Permissible Live Load, in Pounds per Square Foot of Floor Area, for Columns and Foundations, in Hotels, Apartments, Tenements and Lodging Houses and Office Buildings.

	Number of Stories in Building.									
	12	11	10	9	8	7	6	5	4	3
Roof	30	......	......	......	......	......	......	......	......	......
12th Story	50	30	......	......	......	......	......	......	......	......
11th "	35	50	30	......	......	......	......	......	......	......
10th "	25	35	50	30	......	......	......	......	......	......
9th "	20	25	35	50	30	......	......	......	......	......
8th "	15	20	25	35	50	30	......	......	......	......
7th "	10	15	20	25	35	50	30	......	......	......
6th "	10	10	15	20	25	35	50	30	......	......
5th "	5	10	10	15	20	25	35	50	30	......
4th "	5	5	10	10	15	20	25	35	50	30
3d "	5	5	5	5	10	15	20	25	35	50
2d "	0	0	0	5	5	10	15	20	25	35
1st "	0	0	0	0	0	5	5	15	20	25
Totals	210	205	200	195	190	185	180	175	160	140

If the first or any other story is used for a store, hall, or for other business purposes, the full live load of 100 lb. per sq. ft. shall be considered as acting on the supporting columns.

Bearing Capacity of Different Kinds of Soils.

Bearing Material.	New York.	Chicago.	Phila.	Buffalo.	Mpls.	Milw.	Dist. of Colum.	St. Louis.
	Tons per Square Foot.							
Soft clay	1	......	......	......	......	1	1	According to test with a maximum for any soil of 3 tons per sq. ft.
Ordinary clay with wet sand	2	1½	......	......	......	......	2	
Dry clay	3	2¼	......	3½	3½	2	3	
Dry sand	3	2	......	......	3½	2	3	
Hard clay	4	......	3½	......	......	4	4	
Firm coarse sand	4	......	......	......	......	4	4	
Clay and gravel	......	1¾	......	......	......	......	......	
Sand and gravel	......	......	3½	......	......	......	......	
Cemented gravel	......	......	6	......	......	......	......	
Gravel and sand (well cemented)	......	......	......	......	......	8	......	
Rock, through earth (open caissons)	8	......	......	......	......	......	......	
Firm gravel or hard clay (through earth)	10	......	......	......	......	......	......	
Rock, through earth (pneumatic caissons)	15	......	......	......	......	......	......	
Concrete in foundations	......	4	......	4	4	......	......	
Dimension stones in foundations	......	5	......	6	6	......	......	
Dressed foundation stones, in cement mortar	......	7	......	7	7	......	......	
	Tons.							
Maximum pressure on one pile	20*	25	20	25	......	......*	25*	......
	Allowable fiber stress, in pounds per square inch.							
Steel beams in concrete, in foundations	......	16 000	......	16 000	......	......	......	......
Oak timber grillage on piles	......	1 200	......	1 200	......	......	......	......

* Safe sustaining power of one pile, in tons, is:

$$S = \frac{2 \times \text{weight of hammer, in tons} \times \text{height of fall, in feet}}{\text{Penetration, in inches (under last blow)} + 1 \text{ in.}}$$

Bearing Capacity of Materials.

Bearing Material.	Tons per Square Foot.								
	New York.	Chicago.	Phila.	Boston.	Buffalo.	Mpls.	Milw.	Dist. of Colum.	St. Louis.
Concrete: Portland cement 1, sand 2, stone 4	16½	4	......	......	......	......	.....	16½	.....
Concrete: Portland cement 1, sand 2, stone 5	15	......	15	......	......	......	.....	15	.....
Concrete: Rosendale cement 1, sand 2, stone 4	9	......	......	......	......	......	.....	9	.....
Concrete: Rosendale cement 1, sand 2, stone 5	8	......	......	......	......	......	.....	8	.....
Rubble in Portland cement mortar	10	......	10	......	......	......	.....	10	.....
Rubble in Rosendale cement mortar	8	......	......	......	5	5	.....	8	.....
Rubble in lime and cement mortar	7	......	8	......	......	......	.....	7	.....
Rubble in lime mortar	5	......	5	......	......	......	.....	5	.....
Brickwork in Portland cement mortar	18	12½	15	15	12	12	18	18	15
Brickwork in Rosendale cement mortar	15	9	......	......	5-9	5-9	.....	15	.....
Brickwork in lime and cement mortar	11½	......	12	12	......	......	14⅔	11½	11½
Brickwork in lime mortar	8	6½	8	8	3-6	4-6	10⅔	8	8
Granite	72-178	......	......	60	......	......	.....	72-178	.....
Marble and limestone	43-166	......	......	40	......	70	.....	43-166	.....
Sandstone	29-115	......	......	30	......	30	.....	29-115	.....
Bluestone	144	......	......	......	......	......	.....	144	.....
Slate	72	......	......	......	......	......	.....	72	.....
Hard-burned brick in piers, cement mortar	......	......	......	13	......	......	.....	......	.....
Hard-burned brick in piers, lime and cement mortar	......	......	......	10	......	......	.....	......	.....
Hard-burned brick in piers, lime mortar	......	......	......	7	......	......	.....	......	.....

PERMISSIBLE UNIT STRESSES IN MATERIALS.

Material.	Pounds per Square Inch.								
	New York.	Chicago.	Phila.	Boston.	Buffalo.	Mpls.	Milw.	Dist. of Colum.	St. Louis.
Tension.—Rolled steel	16 000	15 000	......	15 000	16 000	16 000	16 000	16 000	......
Cast "	16 000	......	......	......	......	......	......	16 000	......
Medium "	......	......	16 250	......	......	......	......	......	......
Soft "	......	......	14 500	......	......	......	......	......	......
Cast iron	3 000	......	......	......	3 000	......	3 000	3 000	......
Wrought "	......	......	12 500	......	12 000	12 000	......	12 000	......
Yellow pine	1 200	......	1 800	......	......	......	......	1 200	......
White "	800	......	......	......	......	......	......	800	......
Spruce	800	......	1 400	......	......	......	......	800	......
Oak	1 000	......	......	......	......	......	......	1 000	......
Hemlock	600	......	1 000	......	......	......	......	......	......
Compression.—Rolled steel	16 000	15 000	......	12 000	......	......	......	16 000	12 000
Cast steel	16 000	......	......	......	......	......	......	16 000	......
Cast iron	16 000	10 000	11 500	......	13 000	......	......	16 000	......
Steel pins and rivets (bearing)	20 000	20 000	22 000	18 000	15 000	15 000	......	20 000	......
Wrought iron	......	......	8 300	......	......	......	......	12 000	......
Soft steel	......	......	14 500	......	......	......	......	......	......
Medium "	......	......	16 250	......	......	......	......	......	......
Oak (with grain)	900	......	......	......	......	......	......	900	......
" (across ")	800	250	......	250	......	......	......	800	......
Yellow pine (with grain)	1 000	......	750	......	......	......	......	1 000	......
Yellow pine (across grain)	600	250	550	250	......	......	......	600	......
White pine (with grain)	800	......	......	......	......	......	......	800	......
White pine (across grain)	400	150	......	150	......	......	......	400	......
Spruce (with grain)	800	......	500	......	......	......	......	800	......
" (across ")	400	150	300	150	......	......	......	400	......
Locust (with grain)	1 200	......	......	......	......	......	......	1 200	......
" (across ")	1 000	......	......	......	......	......	......	1 000	......
Hemlock (with grain)	500	......	350	......	......	......	......	......	......
" (across ")	500	......	250	......	......	......	......	......	......
Chestnut (with grain)	500	......	......	......	......	......	......	500	......
Chestnut (across grain)	1 000	......	......	......	......	......	......	1 000	......

PERMISSIBLE UNIT STRESSES IN MATERIALS.

Material.	Pounds per Square Inch.								
	New York.	Chicago.	Phila.	Boston.	Buffalo.	Mpls.	Milw.	Dist. of Colum.	St. Louis.
Shear—Steel	9 000	10 000	......	10 000	7 000	7 000	......	9 000	12 000
" Shop rivets and pins	10 000	......	11 000	......	9 000	9 000	9 000	10 000	*9 000
" Field rivets	8 000	......	8 800	......	8 000	8 000	7 500	8 000	7 000
" Field bolts	7 000	......	......	......	......	......	......	7 000	7 000
Soft steel	......	......	8 750	......	......	......	......	......	......
Med. "	......	......	10 000	......	......	......	......	......	......
Wrought iron	......	......	7 500	......	......	......	......	......	......
Cast "	3 000	......	......	......	......	......	......	3 000	......
Yellow pine (with grain)	70	100	100	100	......	......	......	70	......
" " (across ")	500	......	1 120	......	......	......	......	500	......
White " (with ")	40	80	......	80	......	......	......	40	......
" " (across ")	250	......	......	......	......	......	......	250	......
Spruce (with ")	50	80	70	80	......	......	......	50	......
" (across ")	320	......	750	......	......	......	......	320	......
Oak (with ")	100	150	......	150	......	......	......	100	......
" (across ")	600	......	......	......	......	......	......	600	......
Locust (with ")	100	......	......	......	......	......	......	100	......
" (across ")	720	......	......	......	......	......	......	720	......
Hemlock (with ")	40	......	60	......	......	......	......	......	......
" (across ")	275	......	620	......	......	......	......	......	......
Chestnut (" ")	150	......	......	......	......	......	......	......	......

* For Rivets—Pins 12 000 lb. per sq. in.

Permissible Unit Stresses in Materials.

Material.	Pounds per Square Inch.								
	New York.	Chicago.	Phila.	Boston.	Buffalo.	Mpls.	Milw.	Dist. of Colum.	St. Louis.
Bending—Rolled steel beams..	16 000	16 000	......	16 000	16 000	16 000	16 000	16 000	16 000
Rolled steel pins, rivets and bolts	20 000	22 500	......	22 500	......	......	......	20 000	20 000
Riveted steel girders, net section.........	14 000	15 000	......	......	13 500	13 500	12 500	14 000	12 000*
Cast iron (tension)...	3 000	2 500	3 750	2 500	......	......	3 000	3 000	......
Cast iron (compression)................	16 000	......	......	8 000	......	......	13 500	16 000	......
Yellow pine..........	1 200	1 250	1 600	1 250	1 800	......	1 440	1 200	......
White pine...........	800	750	......	750	1 080	1 080	900	800	......
Spruce...............	800	750	1 100	750	......	......	......	800	......
Oak..................	1 000	1 000	......	1 000	1 350	1 350	1 080	1 000	......
Locust...............	1 200	......	......	......	......	......	......	1 200	......
Hemlock..............	600	......	900	......	1 080	1 080	......	......	......
Chestnut.............	800	......	......	......	......	......	......	800	......
Washington fir.......	......	......	......	......	......	1 800	......	......	......
Granite..............	180	......	......	......	......	......	......	......	......
Gneiss...............	150	......	......	......	......	......	......	......	......
Limestone............	150	......	......	......	......	......	......	......	......
Slate................	400	......	......	......	......	......	......	......	......
Marble...............	120	......	......	......	......	......	......	......	......
Sandstone............	100	......	......	......	......	......	......	......	......
Bluestone............	300	......	......	......	......	......	......	......	......
Portland cement 1:2:4	30	......	......	......	......	......	......	......	......
" " 1:2:5	20	......	......	......	......	......	......	......	......
Natural " 1:2:4	16	......	......	......	......	......	......	......	......
" " 1:2:5	10	......	......	......	......	......	......	......	......
Brick (common)......	50	......	......	......	......	......	......	......	......
Brickwork in cement	30	......	......	......	......	......	......	......	......

*Compression Flange (gross section.)

PERMISSIBLE UNIT STRESSES IN MATERIALS.

Material.		Pounds per Square Inch.								
		New York.	Chicago.	Phila.	Boston.	Buffalo.	Mpls.	Milw.	Dist. of Colum.	St. Louis.
Columns:										
Yellow pine	$\frac{L}{D}$ = 10..	820				1 000	1 000	Rectangular Columns $S = 900 \div \left(1 + \frac{L^2}{250\,D^2}\right)$	820	....
	12..	784	1 000		1 000			Round $S = 900 \div \left(1 + \frac{L^2}{185\,D^2}\right)$	784	....
	15..	730							730	....
	20..	640		$S = 750 - 7.5\frac{L}{D}$		$S = 1\,000 - 10\frac{L}{D}$	$S = 1\,000 - 10\frac{L}{D}$		640	....
	25..	550	875		875				550	....
	30..	460							460	....
	40..	...	750		750				...	....
	45..	...	625		625				...	....
	50..	...	500		500				...	....
White pine and spruce	$\frac{L}{D}$ = 10..	650	625		625	700	700	Rectangular Columns $S = 600 \div \left(1 + \frac{L^2}{250\,D^2}\right)$	650	....
	12..	620						Round $S = 600 \div \left(1 + \frac{L^2}{185\,D^2}\right)$	620	....
	15..	575							575	....
	20..	500	500	$S = 500 - 5\frac{L}{D}$	500	$S = 625 - 6\frac{L}{D}$	$S = 625 - 6\frac{L}{D}$		500	....
	25..	425							425	....
	30..	350							350	....
	35..	...	375		375				...	....
	45..	...	250		250				...	....
	50..	...							...	....
Oak	$\frac{L}{D}$ = 10..	730				800	800	Rectangular Columns $S = 800 \div \left(1 + \frac{L^2}{250\,D^2}\right)$	730	....
	12..	696	750		750			Round $S = 800 \div \left(1 + \frac{L^2}{185\,D^2}\right)$	696	....
	15..	645							645	....
	20..	560		$\left(S = 350 - 3.5\frac{L}{D}\text{ for hemlock}\right)$		$S = 750 - 7.5\frac{L}{D}$	$S = 750 - 7.5\frac{L}{D}$		560	....
	25..	475	650		650				475	....
	30..	390							390	....
	40..	...	560		560				...	....
	45..	...	470		470				...	....
	50..	...	375		375				...	....

PERMISSIBLE UNIT STRESSES IN MATERIALS.

Material.	Pounds per Square Inch. New York.	Chicago.	Phila.	Boston.	Buffalo.	Mpls.	Milw.	Dist of Colum.
Columns: Cast iron....	$S = 11\,300 - 30\frac{L}{R}$	10 000 (reduced by Gordon's formula.) $\left(\frac{L}{D} = 20\right.$ is maximum.)	$S = \frac{11\,700}{1 + \frac{L^2}{400\,D^2}}$	See table on page 412.	Rectangular columns $S = 14\,000 \div \left(1 + \frac{L^2}{850\,D^2}\right)$ Round columns $S = 14\,000 \div \left(1 + \frac{L^2}{600\,D^2}\right)$	Rectangular columns $S = 14\,000 \div \left(1 + \frac{L^2}{850\,D^2}\right)$ Round columns $S = 14\,000 \div \left(1 + \frac{L^2}{600\,D^2}\right)$	Rectangular columns $S = 8\,000 \div \left(1 + \frac{L^2}{800\,D^2}\right)$ Round columns $S = 8\,000 \div \left(1 + \frac{L^2}{600\,D^2}\right)$	$S = 11\,300 - 30\frac{L}{R}$
Steel.........	$S = 15\,200 - 58\frac{L}{R}$*	15 000 reduced by approved modern formula.	Soft steel $S = 14\,500 \div \left(1 + \frac{L^2}{13\,500\,R^2}\right)$ Medium steel $S = 16\,250 \div \left(1 + \frac{L^2}{11\,000\,R^2}\right)$	12 000 reduced by approved modern formula.	$\frac{L}{R} > 90$ $S = 17\,100 - 57\frac{L}{R}$ $\frac{L}{R} < 90$ $S = 12\,000$	$\frac{L}{R} > 90$ $S = 17\,100 - 57\frac{L}{R}$ $\frac{L}{R} < 90$ $S = 12\,000$	$S = \frac{15\,000}{1 + \frac{L^2}{36\,000\,R^2}}$	$S = 15\,200 - 58\frac{L}{R}$
Columns: Maximum limit $\frac{L}{R}$..	120 (steel and iron)	....	140	....	....	....	....	120 (steel and iron)
			or					
Maximum limit $\frac{L}{D}$..	30 (timber)	20 (cast iron)	45	....	40 (steel)	40 (steel)	....	30 (timber)
Modulus of elasticity:								
Steel	......	....	....	29 000 000	....	....	....	......
Iron..........	......	....	....	27 000 000	....	....	....	......
White pine....	......	....	....	750 000	....	....	....	......
Spruce	......	....	....	900 000	....	....	....	......
Yellow pine....	......	....	....	1 300 000	....	....	....	......
White oak.....	......	....	....	860 000	....	....	....	......

* For compression members in pin-connected trusses, use 75% of working stresses for columns.

From Boston Building Law.

Working Stresses for Columns of Cast Iron.

$\frac{L}{D}$	Round Columns. S			Rectangular Columns. S		
	Square-faced bearings.	Round and faced bearings.	Round bearings.	Square-faced bearings.	Round and faced bearings.	Round bearings.
1.0	8 480	7 870	7 350	8 810	8 320	7 870
1.5	7 120	6 220	5 530	7 670	6 870	6 220
2.0	5 810	4 810	4 100	6 490	5 580	4 810
2.5	4 710	3 720	3 080	5 420	4 410	3 720
3.0	3 820	2 920	2 360	4 520	3 540	2 920

Note.—In this table L = Length, in feet.

DISCUSSION.

W. B. W. Howe, M. Am. Soc. C. E. (by letter).—This paper is valuable in bringing together and contrasting the practice of various cities in the matter of permissible loads and stresses. There seems to be no good reason why some of these differences should exist, and it is to be hoped the discussion will lead to greater uniformity in this respect. Mr. Howe.

In the matter of determining the bearing power of piles, it does not seem wise to incorporate in a general specification a requirement based upon a formula of uncertain practical value, particularly as the results given by it are confessedly not to be relied upon beyond certain fixed maximum limits.

Long ago the writer reached the conclusion that the actual safe bearing capacity of a pile could not be determined by a general formula, and tests made by him from time to time in different localities seem to bear out the correctness of this conclusion.

If a pile sustained its load by transmitting it entirely to its point, there might be some reason to expect a formula based upon the settlement under the last blow of the hammer to yield approximately correct results, but, as is well known, under certain conditions, nearly the whole supporting power is derived from the friction of the material penetrated upon the sides of the pile, and this friction is not developed until after a considerable period of rest. This being the case, the settlement at the last blow, under such conditions, is not the controlling factor in the ultimate result.

From a set of tests made by the writer some years ago, where the piles were driven in deep alluvial soil, it was found that with a settlement of from 12 to 15 in. at the last blow of a 2 000-lb. hammer falling 20 ft., the pile could be safely loaded to about 200 lb. per sq. ft. of surface of contact, and, in one instance, where the settlement at the last blow was as much as 26 in., the pile carried, without further settlement during the time it was under observation, a live load of 15 000 lb. In this instance the average diameter of the pile was 10 in., and the depth in the ground 40 ft.

Upon another occasion, a pile trestle, in which the piles were driven to a final penetration of 1½ in., the weight of the hammer and the fall being similar, settled badly under the same loading, and continued to settle until the track was taken off, the material penetrated being different in the two cases.

The writer does not believe his experience to have been at all unusual. The method he uses is to test the individual foundation by actually driving test piles, where some positive information cannot be obtained in any other way, and he thinks this the only safe method.

Mr. Worthington.

CHARLES WORTHINGTON, M. AM. SOC. C E. (by letter).—In the writer's opinion, the specifications submitted in this paper are rational and complete, and in them Mr. Schneider has presented a most valuable instrument to those having work of this kind to design.

On the subject of wind pressure: The writer, a while ago, observed the erection of two buildings just across the street from each other, each about sixteen stories high. The framework in one appeared to be thoroughly braced for wind stresses, by deep girders and gussets; the other, apparently, was designed as though there was to be no wind in the vicinity. These buildings have been completed several years, and used for office purposes; one appears to be as good as the other, and it would probably be hard to convince the owners of the lighter structure that they should have spent more money in the steel frame of their building, should have paid more for something hidden entirely from view and apparently unnecessary.

The writer does not mean by this to advocate designing structures of this type without regard to wind forces, but he does believe that there are elements of strength here which are not always considered.

The very large direct loads producing practical continuity in the columns; the accumulated portal effect of many girders properly connected to the columns at top and bottom; a well-chosen system of deep girders, with columns properly located for resisting bending stress; and the great resistance offered by the substantial outside walls, all combine to resist effectually, in most cases, all wind stresses, without requiring any actual increase in the cost of the framework, excepting in gussets or lug angles on girders, and extra rivets for connecting them to the columns.

The writer thinks that, for completeness, the following clause ought to be included.

a.—Rivets in tension members shall be located so as to cut out as little of the section as possible, the rupture of a riveted tension member being considered as equally probable, either through a transverse line of rivet holes or through a diagonal line of rivet holes where the net section does not exceed by 30% the net section along the transverse line. (From Cooper's specifications, modified.)

b.—Tension and compression members shall have connecting rivets placed, as far as possible, on the line passing through the center of gravity of the section, or else grouped symmetrically about the same.

c.—In locating rivets of a group intended to transfer a certain stress, they shall be placed so that the axis of stress, if extended, will pass through the center of gravity of the group of rivets.

d.—Rivets connecting flange angles of girders to the web shall be spaced, at any point in the length of girder, not farther apart (in inches) than the product of the distance (in inches) between the rivet lines in the top and bottom flanges, by the governing value of one

rivet (in shear or bearing, as the case may be), divided by the shear at that point. Mr. Worthington.

The writer also favors using for compression a flat unit stress of 12 500 lb. for values of $\frac{l}{r}$ equal to or less than 60, instead of the variable unit stress given by Mr. Schneider. This simplifies the calculations, and, for such short columns as are ordinarily used in structures of this type, will not affect the final results materially.

Mr. Worcester. J. R. Worcester, M. Am. Soc. C. E. (by letter).—This paper is a valuable contribution to the literature concerning building construction, and, undoubtedly, it will have the effect of improving present practice in a marked degree. At the present time, more steel is wasted in buildings than in any other branch of construction where steel is used. Bridge engineers have reduced designing to a science, so that the steel in a bridge is distributed where it is most needed, and there are few parts of a bridge structure that contain surplus material, but the engineering of steel buildings is in about the same condition of enlightenment that surrounded bridge building thirty years ago, when all parts of a bridge, from stringers to trusses, were designed for the same uniform load per running foot.

The reason for this is, largely, because engineers are not given a chance to use their discretion in the design of buildings, but are obliged to conform with building laws, which must be made so simple that they can be used by men without a technical training. Moreover, in the framing of the laws, it is necessary that they should be so simple that they can be understood by legislators having little interest in matters of construction.

The consideration of the subject by this Society, will, at least, have the effect of furnishing an authority for those interested in the revision of laws to refer to, and is particularly opportune at the present time, because, on account of recent conflagrations, the subject of building-law revision is prominent in many parts of the country.

The author, in dealing so exhaustively with his subject, has opened up a number of questions that should be carefully considered, and it is a question whether he has not gone a little too far in some of his radical recommendations of changes in present practice. The most important question suggested is in the reduction of the uniform load per square foot for floors. While the proposed single concentrations and loads per linear foot for girders will, in most cases, supersede the uniformly distributed load on floors, it does not seem to be wise to reduce this uniformly distributed load to a point below that which is likely to be placed upon any floor. The writer thinks that the author has laid too much stress upon the experiments by Messrs. Blackall and Everett, in Boston. These gentlemen investigated only three office buildings, and it is not likely that in taking three buildings at

Mr. Worcester. random, and making a single examination of each, really maximum conditions would be found. In determining the maximum loads on bridge floors, it would hardly be fair to select three bridges at random, measure up the loads found upon them at a certain time, and consider that the maximum of these three was all that would be necessary to provide for in designing new bridges. To be sure, the load, in the form of office furniture and fittings, might be said to be typical of that in all office buildings, but, in the writer's opinion, every part of every building should have capacity enough to carry the load of a crowd of people. Even in dwelling-houses, all know that, occasionally, companies will assemble sufficient to make a fairly dense crowd in several rooms at the same time. According to the experiments* of Professor L. J. Johnson and Professor C. M. Spofford, it is within the bounds of possibility to get a load exceeding 150 lb. per sq. ft. from a crowd of people, while 80 lb. per sq. ft. represents not more than might be easily assembled at a social gathering. If any parts are designed for 40 lb. per sq. ft., with the usual unit stresses on the material, it is quite possible that the elastic limit might be reached without excessive crowding. To the writer, it seems as if 50 lb. per sq. ft., which has been the loading required for dwellings, hotels, etc., in a number of cities, is low enough, though, with the alternative concentrated loadings, it seems as if this might be applied to office buildings and public assembly rooms, schools, etc.

Taking up the detailed provisions of the specifications, the writer would suggest the following modifications:

Paragraph 6.—The words "are likely to," in the second line, should be changed to "can," and the word "distributed" should be inserted before "live loads," so that the paragraph would read:

"Flat roofs of office buildings, hotels, apartment-houses, etc., which can be loaded by crowds of people, shall be treated as floors, and the same distributed live loads shall be used as specified for hotels and dwelling-houses."

Paragraph 10.—The reduction of live loads on columns should not apply to warehouses which are likely to be loaded on all floors at the same time.

Paragraph 12.—It is an open question whether 30 lb. per sq. ft. is not an excessive wind pressure to allow on city buildings. Possibly, the author intends to allow a certain amount of discretion as to what should be considered "exposed surfaces." At any rate, it seems as if it would be safe not to require special wind bracing in steel frames of buildings of which the least horizontal dimension is as great or greater than the height.

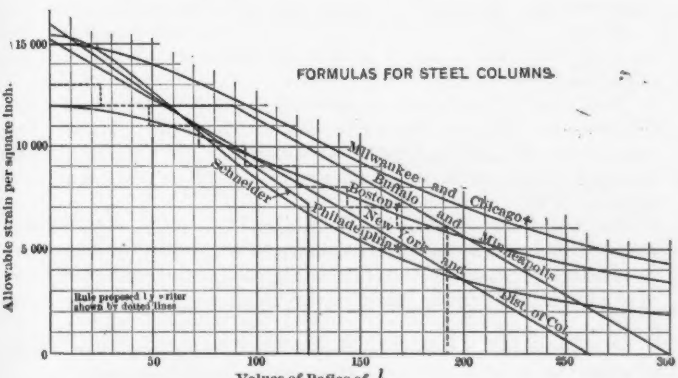
Paragraph 13.—It would be better not to include "wet sand" with "soft clay," as wet sand frequently has much greater supporting

* *Engineering News*, April 14th and May 5th, 1904.

power than 1 ton per sq. ft. The writer would recommend increasing the allowable load for ordinary clay to 3 tons, and for dry sand and dry clay to 4 tons, and for hard clay to 5 tons. Mr. Worcester.

Paragraph 14.—The allowable pressure on hard-burned brick with Portland cement mortar might safely be increased to 15 tons. The pressures allowable for Portland cement concrete seem unnecessarily conservative; 30 tons per sq. ft. is safe with either grade of Portland cement concrete mentioned.

Paragraph 15.—The pressures under wall-plates on brickwork with cement mortar, or rubble masonry with cement mortar, might be increased to 200 lb., on Portland cement concrete to 400 lb., and on first-class sandstone to 400 lb. On granite, 800 lb. is all right, and it



$\ast$ Average between values for medium and soft steel.

$+$ Using Rankine's reduction formula.

FIG. 3.

seems as if it were advisable to have one pressure specified instead of upper and lower limits.

Paragraph 19.—In adopting the bridge formula for compressive strains in columns, the author proposes very different units from those commonly allowed. The diagram, Fig. 3, shows the strain per square inch allowed by various formulas. The ordinates give the unit strain, and the abscissas proportions l to r, between 0 and 300. It is evident from this diagram that there is a wide divergence of opinion as to the proper allowable units, and it seems to the writer that, where such great variations are found between different authorities, it is absurd to split hairs by calculating different unit strains for each variation of the value of l to r. As an alternative rule for compression, which

Mr. Worcester. would give in every case safe results and be much easier of application, the following is suggested:

"Columns may be used with a ratio of l to r not exceeding 16, l being expressed in feet and r in inches. The unit strain allowed shall be 13 000 lb. per sq. in. for $\frac{l}{r}$ from 2 to 4, 12 000 lb. from 4 to 6, with a decrease of 1,000 lb. for each succeeding increase of 2 in the ratio of l to r."

The result of this rule is also plotted in Fig. 3, and is shown by the stepped line. In the use of this rule, practically all the columns in the same tier of a building would be calculated for the same unit strain, and this would be a round number, whereas a close figuring by any other rule would require a large majority of the columns to be calculated separately. The economy of this is readily appreciated by those who have had experience in designing building framework.

Paragraph 28.—The author's limit for compression members, 125 times the least radius, would in many cases necessitate a large increase of material, particularly in roof trusses.

Paragraph 29.—The rule of allowing the web to be calculated in the flange area of plate girders is unsafe without the provision that, where thus used, splices must be made at points where the total flange section is not required, or special provision must be made in splicing to transmit bending moments.

Paragraph 33.—A limit of one-fifteenth of the depth would in some cases infringe on architectural clearance in a way that would be very troublesome. The rule should be arranged so as to provide for beams of shallower proportions than this, by using properly increased unit strains.

Paragraph 35.—As cast iron is allowable for column bases, even if nowhere else, a limiting tensile fiber strain should be specified.

Paragraph 37.—It would be interesting to know why the author considers it necessary to use, for wooden columns, a formula based on Gordon's, while, in the case of steel columns, he applies a straight-line formula. There is even more variety in requirements for wooden columns than there is for steel, as shown by Fig. 4, in which are plotted the strains allowed on yellow pine by the cities quoted in the Appendix. The writer would suggest as a substitute the following:

"Columns may be used with a length not exceeding 50 times the least dimension. The unit strain for lengths up to 10 times the least dimension shall be as given in Paragraph 36, with a reduction of 100 lb. per sq. in. for every increase of 10 in the ratio of length to least dimension."

Paragraph 39.—It would be in the interests of permanence of construction if the limiting thickness of the material in the outside walls were made $\frac{5}{16}$ in., and where the protection is likely to be not more than 4 in., or, at most, 8 in., of brick, it seems as if this requirement should be insisted upon.

Paragraph 45.—A rule for the placing of tie-rods, based upon nothing except the depth of the beams, is extremely rough. In many kinds of construction where flat slabs are used, for instance, where the slab rests on top of the beam, tie-rods are practically superfluous; while, on the other hand, where segmental brick arches are used for floor construction, it is quite possible that the spacing of eight times the depth of the beam would be altogether too great for small rods. To make this paragraph correspond with the thoroughness of the remainder of the specification, it seems as if a rule should be given by which the tie-rods could be calculated properly. The only uncertainty on this point is as to how much dependence should be placed upon the support afforded by the arches in adjoining bays. It is unnecessary to calculate on the rods in one bay resisting the entire thrust from the arch of that bay, provided the bay is one of a series. Mr. Worcester.

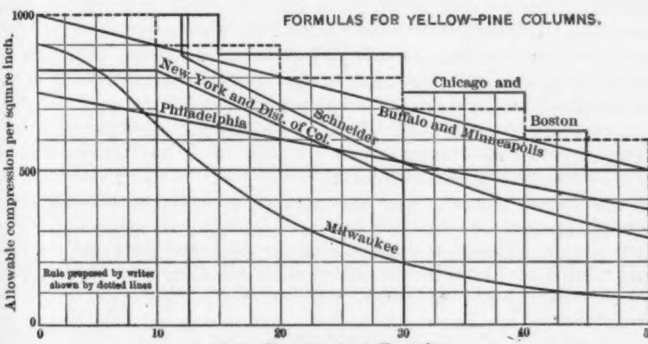


FIG. 4.

As a simple rule, which is approximately correct, it is suggested that rods sufficient to resist half the total thrust should be provided.

Paragraph 49.—"Connection angles" may mean a variety of styles of connections, some of which are not desirable.

Paragraph 54.—The question might be asked: What should be provided in the way of splices if there is no bending strain?

Paragraph 57.—In many instances trussed purlins are economical and good practice. The reason for prohibiting them should be further explained.

Paragraph 61.—In members composed of two angles only, the maximum pitch, 6 in., is altogether too short, and does more harm than good.

Paragraph 80.—The application of this to building construction would be so rare as to make the clause superfluous.

Mr. O'Brien. JOSEPH H. O'BRIEN, ASSOC. M. AM. SOC. C. E.—This paper has proved so interesting that the speaker has taken occasion to review it quite carefully.

In what follows it is not the speaker's purpose to criticize the author, but merely to call attention to some points which seem to need more explanation than has been accorded them.

Mr. Schneider advises that all floor beams for floors of office buildings be tested for the maximum effect thereon of a concentrated load of 5 000 lb. (equivalent to the weight of the larger portable office safes). It is evident that if a safe weighing 5 000 lb. has any appreciable area, one floor beam (under usual conditions) would not support the entire weight; therefore, 5 000 lb. will be a liberal and not an actual value, if it is assumed to be concentrated on one floor beam.

Similarly, in regard to the load of 326 lb. per lin. ft. determined from the weight of a filled plan case, the author assumes the entire load on one floor beam, although he gives a width of 31 in. for his plan case. Of course, if someone should pile the cases two high, the author's assumed loading would be increased. However, his assumption of the entire 326 lb. per lin. ft. on one beam is so liberal that, doubtless, it would cover the possible condition just referred to.

The speaker's contention is regard to distribution of load is admitted by the author in the case of the book cases, but not in the case of the 5 000-lb. concentrated load.

The author states in the last paragraph of the section entitled "Wind Loads":

"In proportioning the members of the structure for these temporary wind strains, it is permissible to allow a higher unit strain than for permanent work, say 20 000 lb. per sq. in., or about two-thirds of the elastic limit."

The speaker has been accustomed to the use of some such liberal unit strain for wind bracing only, using the usual value for permanently loaded members, and he thinks this practice to be in conformity with a strict interpretation of the New York Building Code. The author would permit an increased unit value for strains due to wind on the entire structure. To carry out the author's suggestion it is only necessary to reduce the wind strains on permanently loaded parts of the structure in the ratio of the larger to the smaller unit value. For example, if the permissible unit strain for dead loads is 15 000 lb. per sq. in., and the supporting member considered sustains a wind strain at intervals equal to 60 000 lb., then fifteen-twentieths or three-fourths of 60 000 lb., namely, 45 000 lb., could be used as the wind strain to be added to the dead-load strains on the member, giving a total strain which, when divided by the working unit for dead load (15 000 lb.) would satisfy the author's recommendation.

If the author's suggestion in this particular be generally adopted it will be necessary to make the building laws more explicit.

Mr. O'Brien. In Paragraph 17 of the specifications, the author gives an item, "Shear, on plate-girder web (gross section) 10 000 lb. per sq. in."

Before coming to New York City the speaker had been accustomed to use the gross section in determining the girder webs for building work, but when engaged in writing designers' specifications for some New York City work, the design of which he is directing, he made careful inquiry in regard to New York City practice in this particular, and found that, even those concerns accustomed to exceptionally economical frames used the net section in determining plate-girder webs. To be sure, the reduced strain in the gross section of the web, if web rivet holes are deducted, makes the girder proof against buckling in most cases and, except where good practice requires stiffeners for other reasons, they could be omitted.

It is so seldom, however, that increased web-plate section *versus* stiffeners comes up for consideration, as related solely to the plate-buckling value, that the practice of deducting web rivets seems to the speaker to be an outgrowth of some other consideration.

In searching for this other consideration, the speaker was confronted with the possibility of loose rivets in the end-bearing stiffeners, but, as the reaction gets into the girder through the medium of the rivets in the end connection or bearing, and the vertical forces in the girder reach the abutment through the same medium, the loose rivets would cause an increased strain in the tight ones, and as these rivets, if capable of resisting failure from such a strain, would, if the plate tended to shear, meet the resistance of two vertical shearing surfaces in the plate, it appears that, for consideration of purely vertical shear, particularly with machine rivets (which are seldom loose) in the end stiffeners or connection angles, it is not necessary to deduct rivet holes in determining the web section.

Another point is that, possibly, in the case of a girder riveted between two columns, deflection would cause a tension in the web coincident with vertical shear, resulting in a tearing action which might cause failure at the end-riveted connection.

The speaker is convinced, however, that it is unnecessary, except perhaps for heavy railroad bridges (where very great rigidity is required), to use the net section in designing plate-girder webs.

Mr. Seaman. HENRY B. SEAMAN, M. AM. SOC. C. E.—This paper is so thorough in its preparation that there is little room for criticism, but there are a few points of difference in practice to which attention might be called.

The provision of the New York Building Law (Paragraph 10), that the loads on the columns of buildings more than five stories high, be reduced, by considering that the lower floors carry the lightest loads, seems to give an erroneous interpretation to the original purpose of the modification from the full load. It provides that floors nearer

Mr. Seaman. the roof may be considered fully loaded, while those nearer the street would be considered only half loaded. This is contrary to probable conditions. The floors nearer the street would naturally be loaded first, and, in a crowded building, would be more likely to remain fully loaded than the floors above.

The purpose of the law would seem to be to provide for the reduction of the average simultaneous load of all floors, rather than merely the unequal loading of the different floors. The speaker, therefore, would suggest the following revision of the wording of Paragraph 10.

"For buildings more than five stories in height, these live loads may be reduced as follows:

"For columns supporting the roof and top floor, no reduction.

"For columns supporting each succeeding floor, a reduction of 5% of the total live load may be made, until 50% is reached, which reduced load shall be used for the columns supporting all remaining floors."

This will give slightly decreased sections, but it will make ample provision for safety, and, very evidently, was the original purpose of these laws.

The provision for uniform dead load pressures on the foundations is an excellent one.

The use of the empirical right-line column formula, instead of the rational one of Rankine, is unfortunate, although this form has come into quite common use. The empirical is not more accurate than the rational form, and is of more limited application, as shown by Paragraph 28. The two forms are usually reduced to tables, or diagrams, and are equally convenient. The step from the rational to the empirical seems to be a step in the wrong direction.

The inconsistency of its adoption here is shown in Paragraph 37, where the Rankine form is used for timber.

The provision (Paragraph 25) that connections subject to alternate strains "shall be proportioned for the strain giving the largest section," does not seem to be sufficient. Reversal of strain is the greatest cause of deterioration in old structures. As soon as the strain in a member is relieved, or reversed, it gives opportunity for wear, and should require greater section than if the strain acted continuously in one direction and if it were constantly applied.

It is quite a common practice to allow one-sixth of the gross section of the web of plate girders to be available as flange area. The plate girder has an advantage in strength and stiffness over other forms of built beams, and, to restrict the use of web area to one-eighth (Paragraph 29) seems to be an unnecessary refinement, in this class of work.

The provision (Paragraph 49) that floor girders shall be riveted to columns by means of connection angles, implies that shelf-angle supports, alone, are not sufficient. It would be well, also, to avoid the

use of bolts wherever practicable. Shelf-angle and bolt construction, now in quite common use, has little advantage over cast-iron column construction, at present so generally, though perhaps unjustly, condemned. Under these loose conditions, the wrought-iron column has not even the advantage of cast iron as protection in case of fire. It might be better, however, to specify that floor girders shall be preferably riveted to columns by web connections, since the connection to a Z-bar column, by making a splice connection of the diaphragm through the center of the column, to the web plate of the girder, makes an exceedingly rigid connection, especially desirable where the ironwork is expected to resist vibrations. Mr. Seaman.

Under "Material and Workmanship," Paragraph 12 provides for modification in elongation, of material less than $\frac{5}{16}$ in., and more than $\frac{3}{4}$ in. in thickness. This might well be omitted. Material more than $\frac{3}{4}$ in. thick should not be used, and it is not worth while to make special provision for material less than $\frac{5}{16}$ in. thick. Thin material is generally used for fillers, and need not be tested. Concessions in required elongation of thick material usually mean that it receives less work in manufacture, and should receive less strain in design. It is a convenience in manufacture, but its use should be avoided. The special case of pins and rollers is provided for elsewhere.

The use of pneumatic hammers, instead of hand riveting, insures much better work than formerly. Provision should also be made for the use of oil rivet-heaters, instead of the hand forge. When these were first introduced upon the New York Subway, several years ago, they both met with considerable opposition; but they won their way into favor, as being both economical and efficient, and have since come into very general use in the field. The labor unions object to the use of the oil rivet-heater, because one man may supply several gangs, but these heaters are so superior to the hand forge that their use is justified, if for only one gang

The provision that, in cutting out rivets, "great" care shall be taken not to injure adjoining metal, is a dangerous clause to place in the hands of an inexperienced or impractical engineer. If a well-heated rivet is used to replace the one removed, ordinary care would be sufficient, as the metal is not likely to be injured seriously.

The last eleven words of Paragraph 65, "unless he is not responsible for the design of the work," should be omitted, as a faulty design is always the designer's responsibility, and should receive protest before testing.

In closing, the speaker wishes to offer his thanks and his congratulations for a very well-considered and exhaustive paper.

AUGUSTUS SMITH, M. AM. SOC. C. E.—Mr. Schneider's opportune paper on the structural design of buildings discusses the present practice so concisely, and, in the comparative tables compiled from Mr. Smith.

Mr. Smith. the building ordinances of various cities, discloses such a surprising diversity of opinion as to loads and allowable stresses, that it will undoubtedly do much to lead discussion on matters where opinions seem to differ so widely, and to crystallize the thought of constructors in this line of work, so as to produce a more uniform practice.

It is perhaps logical that the building laws of different cities should prescribe different loads within certain limits, depending upon local conditions, but surely the safe stresses for materials of construction need not vary for different localities.

The author very pertinently calls attention to the rational design of floors by considering the effect of a concentrated load, as well as a uniformly distributed load, and proportioning the floor joists and girders to meet the most severe condition. This method of designing is highly desirable, for all the reasons pointed out by the author, and for the further reason, which he did not mention, that it would be a more complicated method than the one now in vogue, under which every architect's office boy thinks he is competent to pick out the right-sized beams from a rolling-mill handbook, and thus might lead to the more general employment of competent engineers to design the framework of steel buildings, where mathematical computations are desirable.

Another point which the author has brought out very forcibly is the theory of designing foundations on compressible soils, with the area of the foundation made proportional to the dead load, considering the live load only as a factor in determining the allowable unit pressure on the soil. This is very scientific, and should lead to good results.

The emphasis laid by the author on the importance of considering wind pressure should do much to offset the carelessness in this respect that is manifested by many designers of buildings of moderate height.

Surely the framers of the present Building Laws of the City of New York overlooked such structures as pier sheds and freight sheds when they drew up Section 140, which ends as follows:

"In buildings under 100 ft. in height, provided the height does not exceed four times the average width of the base, the wind pressure may be disregarded."

At the time these Building Laws were being discussed, the speaker attended a "public hearing," arranged by the Board of Aldermen for the purpose of entertaining any objections that might be made to the proposed Law, and argued quite at length on the defect in this section on wind pressure. Unfortunately, a delegate from the Plumbers' Union was next heard and made such a fuss about certain provisions of the law requiring less assistance from the plumbers than might have been called for that the defect pointed out by the speaker was entirely forgotten.

It is fortunate that Mr. Schneider has brought up the question of wind pressure in the way he has. As stated at the beginning of his paper, he invites discussion and criticism of the proposed specifications and of the many new statements and figures contained therein. Mr. Smith.

Confining this discussion to "Part I—Design," the speaker has the following suggestions and criticisms to offer on the subject matter of the following paragraphs:

Paragraph 3.—This paragraph proposes a new requirement, for loads in buildings, which is intended to be more nearly in accordance with the actual loads which the buildings will be called upon to sustain. In office buildings, a proposed uniformly distributed load of 40 lb. per sq. ft. of floor is called for. The average of the requirements of eight cities, in this respect, as given in the Appendix, is 82 lb. per sq. ft.

The application of Paragraph 3 will show, as pointed out by the author, that in most cases the floor would be designed for the concentrated loads and loads per linear foot on girders, whether the distributed load were taken at 40 or 80 lb. per sq. ft. of floor, and the concentrated loads and loads per linear foot seem to have been chosen very wisely. Therefore, the following remarks about the distributed load are somewhat academic. Nevertheless, the light distributed loads prescribed for office buildings, assembly rooms with fixed seats, and ordinary stores, at first glance look like a radical departure from present practice, and seem to the speaker to have a bad "moral effect." For instance, the architect's office boy may still venture on the problem and get his figures tangled up so that a floor may be designed for 40 lb. per sq. ft. instead of for the concentrated loads prescribed by the table, and it is easier and safer to avoid such a mistake, by prescribing a heavier distributed load, than it would be to detect the mistake after the plans were out.

The real reform aimed at by revising the distributed loads in Paragraph 3 is probably in the design of columns, as would appear later by the provisions of Paragraph 9. This reform, however, could doubtless be accomplished in some other way.

A distinction is made in Paragraph 3 for distributed load between assembly rooms having fixed seats and assembly rooms without fixed seats. To the speaker it does not seem proper to make this distinction, because a dancing floor is frequently constructed over the tops of the seats in such rooms, and, even when the seats are used, an audience is quite apt to stamp in unison, which would have nearly the same effect on the floor as dancing or marching would have.

Again, ordinary stores or light manufacturing buildings would seem to involve very similar loading to office buildings, and 40 lb. for this service seems to be very light. The average of the requirements of nine cities given in the Appendix is 119 lb.

Mr. Smith. The speaker agrees with the author that, in many cases, the loads assumed for floors have been excessive, and thinks that all the author has said about reducing the load on columns and on foundations is sound, but when there is so much money available for ornamentation in office buildings, there does not seem to be sufficient reason for lightening the floor construction in buildings of this class as much as Paragraph 3 might allow in certain cases.

In dwellings, on the other hand, there is a positive advantage in economizing the design so that steel-framed buildings may be within the scope of the usual appropriation. The only reason why steel-framed dwellings are not more generally adopted is the higher cost, and the loading prescribed in Paragraph 3 will do something to reduce this and thus encourage the construction of dwellings of this class.

The speaker would suggest, therefore, that office buildings, assembly rooms (with or without fixed seats), and ordinary stores and light manufacturing buildings, should be grouped together and designed for a distributed floor load of not less than 75 or 80 lb. per sq. ft., with the further provision for concentrated loads and girder loads given in Paragraph 3.

Paragraphs 7 and 8.—Why should not the dead load of roofs be calculated as provided for floors in Paragraphs 1 and 2, and only the live loads discussed in Paragraphs 7 and 8?

For the past eight years, the speaker, with unvarying satisfaction to himself and his clients, has used the general specifications for roofs and iron buildings, by Charles Evan Fowler, M. Am. Soc. C. E., of which a fourth revised edition was issued in 1901. The loadings of Fowler's specifications, both for wind and snow, seem to be preferable to the loadings given in Paragraphs 7 and 8.

Paragraph 9.—The distributed loads prescribed in Paragraph 3 were doubtless selected with the design of the columns in view, and, if changed as suggested by the speaker, would involve the rewording of this paragraph.

Paragraph 12.—A wind pressure of 30 lb. per sq. ft. is called for in the New York Building Laws, for buildings more than 100 ft. high, with an allowable unit stress of 50% more than for dead or live loads. Fowler gives 20 lb. for buildings less than 20 ft. high and 30 lb. for buildings 60 ft. high, with no extra allowable unit stress.

The author prescribes a wind pressure of 30 lb. per sq. ft., with an excess of 25% in the allowable unit stresses, and provides further that the framework shall be figured as an independent structure and designed to resist wind without walls, partitions or floors. It is obviously true that the wind pressure cannot be quite as great near the ground as it is some distance above it, and it is also probably true that large areas cannot be subjected to the same pressure per square foot as a small area would be. For instance, the flat side of a building

must have a wedge of dead air in front of it, which would probably Mr. Smith. have the effect of reducing the total wind pressure on the projected area perhaps one-half. On the other hand, the small areas of the frame of a structure do not have this advantage, and, on these small areas, would it not be better to take 50 lb. per sq. ft., as it is commonly specified for wind pressure on bridges? It would appear, also, to make simpler computations if the assumed wind pressure on the side of the building were reduced and no additional unit stress allowed in the frame, which is the method of computation adopted by Mr. Fowler.

Paragraph 13.—The permissible pressure on foundations is so difficult to describe in general specifications and, in a certain sense, being not quite relevant to the design of the building itself, that it might be advisable to omit Paragraph 13 altogether. If, however, some specification for pressure is desirable, the depth of the foundation should enter as a factor. To borrow some words from Naval Architecture, it is apparent that the buoyancy of the soil is a function of the depth of the displacement of the building or other structure.

Paragraph 14.—It is to be noted that the author has given Portland cement concrete considerably higher value than bricks or rubble masonry laid in Portland cement mortar. If the allowable values given by the Building Laws of the City of New York are correct, the proposed value of hard-burned brick and rubble in Portland cement mortar, as given by the author, would seem to be low.

Paragraph 16.—No formula has been suggested for determining the bearing power of piles driven by a steam hammer. The speaker is not prepared to suggest any formula for this class of work, but would call attention to the desirability of one.

Paragraph 21.—The late J. B. Johnson, M. Am. Soc. C. E., gives for the pressure on expansion rollers: $P = 1\,200\sqrt{d}$. If d were taken as 1 in., the author's formula would give one-half of Johnson's allowable load, but if d were taken equal to 4 in., the allowable pressure would be the same by each formula. From a superficial inspection, it would seem to be more logical to take the pressure directly proportional to the diameter than to the square root of the diameter.

Paragraph 22.—It seems to the speaker unsound to treat a combination of transverse loading with tension or compression by the same formula. Certainly, transverse loading of a compression member is more dangerous to the safety of a structure than the transverse loading of a tension member.

Paragraph 29.—The speaker would prefer to have the second sentence read:

"The compression flange shall have at least the gross sectional area of the tension flange, etc."

Paragraph 30.—The speaker would suggest adding the words,

Mr. Smith. "and shall preferably be designed as a strut with transverse loading with adequate moment of inertia laterally." The speaker happens at the present time to be strengthening a crane run-way in which the length of the compression flange was 22 times its width, but in which, if the flange had been one-twentieth of the length of the girder, the result would probably not have been satisfactory.

Paragraph 33.—The limiting depth of roof purlins, if continuous over more than one span, may be less than one-thirtieth of the span and give satisfactory results. The speaker's practice for continuous purlins over three supports, has been:

4-in.	purlins	for	12-ft.	span,	
5 "	"	"	14 "	"	
6 "	"	"	16 "	"	
7 "	"	"	18 "	"	and
8 "	"	"	20 "	"	

The last size of purlin is one-thirtieth of the span, but the others are less.

Paragraph 42.—The speaker would suggest making this paragraph read:

"The strength of connections shall be sufficient to develop the full strength of the member, except in the case of shapes used in tension, or where extra material has been used to reduce deflection or distortion."

Paragraph 45.—The speaker would call attention to the fact that $\frac{1}{30}$ in. per foot of span for a fire-proof floor is a very considerable deflection to allow if the load producing it is apt to occur frequently.

Paragraph 49.—It would be wise to add to this paragraph:

"If shelf angles or other supports are calculated to take any share of the load imposed by the end of a beam or plate girder, the beam or girder shall be riveted securely to the shelf angle or other support."

The object of this requirement is to assure the bearing on the shelf angle that was assumed in the calculations.

Paragraph 56.—The provisions of this paragraph are frequently difficult to meet, in the case of the intersection of the top and bottom chords of flat triangular roof trusses, without making the depth of the truss over the supporting column so shallow as not to provide sufficient metal to take care of the shear. It would be desirable to add, to take care of this and other similar cases, the words:

"And when this is impracticable, the resulting eccentricity must be computed and provided for."

Paragraph 57.—In the case of long roofs, such as pier sheds, it is not necessary that all the roof trusses shall be braced in pairs, for the reason that the purlins or roof planking can be relied upon to sustain intermediate trusses, if, at suitable intervals, a pair of roof trusses be braced together. Perhaps it would be sufficient to obtain good construction if this paragraph were made to read:

"At least 50% of the roof trusses of a building shall be braced together in pairs at suitable intervals in the plane of the top chord, if the roof covering is carried on purlins or jack-rafters." Mr. Smith.

If roof planking be secured directly to the trusses, which is frequently the case, the planking can be relied upon for sufficient bracing.

There would seem to be no gain in bracing the bottom chords of the roof trusses, unless in special cases it should be necessary to take care of the wind pressure against the sides or ends of the building by bracing in the plane of the bottom chord. Roof trusses of the old style, with round wrought-iron rods for tension members, were not braced in the plane of the bottom chord, and they seem to get along quite as well as trusses of the new style, built of angle iron, where the convenience of the connection seems to tempt the designer to add a lot of bracing.

The bottom chord of a roof truss is in tension, and, necessarily, takes the shortest line between the points of support, and will stay there, in consequence of the load on the truss, without any bracing. The requirements of Paragraph 57 would seem to call for unnecessary bracing.

Paragraph 79.—Expansion rollers, considerably less than 4 in. in diameter, would seem to have a legitimate field for use under such girders and trusses as are apt to be found in building design.

Paragraph 81.—The wording of this paragraph would indicate that columns not strained in tension at their base need not be anchored to the foundations. It has always seemed to the speaker advisable to anchor, or at least dowel, columns to foundations, whether or not they are strained in tension, for the reason that changes of temperature or minute vibrations, by moving the base of the column, always in the direction of the least resistance, would tend to make it "creep" off the foundation, and, although this would be impossible in the case of a large building supported by many columns and sunk some distance in the ground, it would possibly tend to set up transverse stresses in the unanchored columns to keep them in place on their foundations.

R. D. Coombs, Jr., Assoc. M. Am. Soc. C. E. (by letter).—The writer does not think that the 40 lb. per sq. ft. uniform live load, for hotels, theaters, schools, etc., in the author's very timely specifications, is high enough. At least, it would seem that a greater value should be used in hallways, lobbies and assembly rooms, of theaters and college buildings. If the calculated load, 156 lb. per sq. in., obtained by Professor Spofford, did not require uncomfortable crowding, then much higher loads than 40 lb. must frequently occur. Mr. Coombs.

Students leaving a classroom, coincident with the arrival of the coming class, would increase the load carried by the hallway beams

Mr. Coombs. and girders to more than this figure. In class "rushes," whether impromptu or premeditated, coming under the writer's observation while in college, the men were "very uncomfortably" crowded over a hall area of about 300 sq. ft.

With the possibility of similar loading, it would seem desirable to specify a live load of 100 lb. per sq. ft. for beams and girders.

Paragraph 65 of Part II.—This paragraph, as worded, would relieve the contractor of the cost of members which might be faulty in design and deficient in unit strength; whereas the latter consideration should cause the rejection of the material. As a modification, it is suggested that the paragraph be worded:

"If it does not develop the specified unit stress at the point of maximum stress, it will be considered rejected material and be solely at the cost of the contractor."

Mr. Llewellyn. F. T. Llewellyn, Assoc. M. Am. Soc. C. E.—This paper forms a valuable contribution to engineering literature, particularly because it deals with a branch which is of the utmost importance to the capitalist, the engineer, and the occupants of buildings, but which, strangely enough, has received much less critical investigation and is based on a more variable practice to-day than almost any other line, unless it be that of reinforced concrete. In both these subjects engineers seem to have reverted to those old Hebrew times when every man did that which was right in his own eyes, and naturally wrong in his neighbor's. The ingenious methods proposed by Mr. Schneider must tend to bring every man and his neighbor into greater uniformity, and along lines that seem self-evidently reasonable. There may be difference of opinion regarding the precise figures to apply as floor loads, working stresses, shop practice, or quality of material, but the method of proportioning the structure, from the floor joists down to the foundations, is so logical, and in line with what has proved proper in bridge construction, that it promises to result in greater conformity and security, and with better disposition of material, at probably no greater cost. It is to be regretted that these specifications could not receive the test of actual use as a basis for more intelligent criticism at this time.

The particular feature of the specifications which the speaker proposes to discuss is their practical result, which appeals to those specializing on structural work. The floor joists would be generally somewhat heavier than in existing practice; short beams would be considerably heavier, and floor girders would be somewhat lighter. These modifications would reduce the total thickness of floors, and minimize projecting girders, which, in hotels and loft buildings, where partition arrangements are subject to change, is very desirable.

Unduly light coping and other connections would be eliminated, resulting, not only in greater strength under concentrated floor loads,

but also allowing more security to the erector in the way of derrick fastenings and guy-line hitches, for the man on the job will hitch to the handiest piece, which is also generally the smallest. Increased uniformity in coping connections would be feasible, permitting the use of multiple punches, and facilitating the preparation of shop details. There would be less variety in the sizes of beams required, thereby securing quicker rollings of material at the mills. There would be less violent changes in the column sections from story to story, to the lasting happiness of the shop draftsman, and also permitting fewer sizes of fire-proofing and masonry. The proposed method of proportioning foundations seems to be admirable, especially where rock is not reached. Mr. Llewellyn.

There is an apparent discrepancy between the allowed use of Bessemer steel in Paragraph 2 of Part II and the chemical requirements of Paragraph 3.

In Paragraph 1 of Part II cast iron is allowed for column bases and bearing plates, and in Paragraph 35 of Part I its allowable unit stress in compression is given; but, in proportioning the thickness of such plates, bending must be calculated, and the allowable unit stress in tension should be inserted. It is suggested that 3 000 lb. per sq. in. would be proper, or a combined bending stress of 5 000 lb. per sq. in., on the average cross-section between ribs, would allow ready calculation, with fair, if not accurate, results.

The most serious omission, however, occurs in Paragraph 21 of Part II. The specifications allow the use of cast iron for base plates, but do not stipulate closely enough the necessary safeguards, and that, too, in items which sustain the entire weight of the building. Provision is made for tests and finish, but there is silence regarding the most important feature of cast iron, namely, its inspection at every stage, and particularly an early morning inspection of castings, poured over-night, while being shaken out. One great advantage of rolled steel is its accessibility to inspection at every stage, and cast iron, if used at all, should be similarly accessible. To illustrate the need of this inspection, one need but glance at the advertising pages of any foundry journal, where such names as "Smooth-it-over," "Pile-it-on," or "Filler-up" are given to substances sold for the purpose of concealing defects in castings. The foundry-man welcomes visitors to inspect the mysterious methods of sand or sweep moulding, or to witness the sprays of sparks that fly from the molten metal while a heat is being taken off at night, but one should see the result in the "cold gray dawn of the morning after," before it can be doctored.

During the middle ages progress was slow, for the reason that all crafts were secret. The advance in the modern use of steel is due largely to the opposite policy, and the facilities for inspecting the

Mr. Llewellyn. product at all stages, not only ensure its reliability, but offer opportunities for each steel master to improve on his neighbor's methods; nor have they been slow to do so. One reason why there is to-day so much doubt as to the safety of cast iron (an excellent material in its proper place) is because short-sighted iron-founders have attempted to force its use where it has no place, and have bolstered up their claims by statements which cannot be investigated on account of their secretive policy in reference to its most important period, namely, just after cooling. Until this attitude, a relic of medieval barbarism, is removed, it will not be possible to estimate properly the real value of cast iron as an engineering material.

Mr. Cooper. THEODORE COOPER, M. AM. SOC. C. E. (by letter).—It is very desirable that specifications for the structural features of buildings should be brought into some general uniformity based on technical common sense. Of course, absolute uniformity is not to be expected, for even experts are not yet in harmony as to the minor details of any class of construction. Mr. Schneider has done a good work in bringing so important a subject up for consideration and discussion.

If the discussion tends to produce any substantial agreement as to live loads to be used for the various cases of everyday practice, as it should, a great advance will have been made in building practice. On the subjects, unit strains, details of construction, shop practice and material specifications, there are less differences, differences which it would be useless to discuss until there is a reasonable agreement as to the live loads which are to be used.

In practice, it is the concentrated loads which determine the strength or safety of the floors and building. Uniform loads, sufficiently high to cover the concentrated loads, produce wasteful construction, without any compensating benefit to the building.

The method proposed by Mr. Schneider, adopting a uniform load sufficient to cover all cases, where the load may be uniformly distributed, and then supplementing this with a concentrated load to provide for any excessive local loadings, is in the line of economic and therefore good engineering.

The distributed load of 40 lb. adopted by the author for people and ordinary fittings in rooms and offices is, in the writer's opinion, a liberal allowance. Rooms are not loaded by dropping the last possible person into the seething mass below by means of tackle, as has been done, to determine the weight of crowds.

In an assembly room of any kind, great local concentration of people may be caused by a fire, fight, or panic, yet the load over the whole floor will not be increased. Most people have experienced the discomfort of a crowded Elevated Railroad car, when not another person could be squeezed inside of the gates. Such a crowd numbering

about 120 persons and not weighing more than 18 000 lb. is contained in a space of about 400 sq. ft., including platforms, or 45 lb. per sq. ft. Mr. Cooper.

A weight of this kind would not be expected in living or office rooms, theaters, churches, schools, armories, ballrooms, etc., over the whole floor. A popular reception or a panic might produce this, or a somewhat larger loading in the aisles or corridors, or on the stairways, but this would be taken care of by the concentrated load.

The author has allowed for 100% impact and vibration, or has increased his uniform load to 80 lb. for ballrooms, drillrooms, etc. This is liberal, for when people are packed in so as to weigh 40 lb. per sq. ft. of floor, there will not be much marching or dancing. Instead of this allowance of 100% in all cases, it would be better and more just to make the allowance variable with the dead weight of the floor, as heavy solid floors should have an advantage over those of light weight.

The concentrated loads adopted by the author appear to have been well selected for the several cases.

Paragraph 13.—In this paragraph the word "permissible" should be omitted, and the words "Pressure on foundations not to exceed" used instead. It is not safe to define the permissible pressures on a foundation solely on a general classification of soils. Limiting pressures may be specified, subject to reduction by local experience or examination.

The classifying of soft clay and wet sand together must be a typographical error.

Paragraph 37.—The straight-line formulas for timber, as deduced from the Watertown tests on timber, should be used, as a simpler form and based on actual tests.

Henry W. Post, M. Am. Soc. C. E. (by letter).—It is to be hoped that the presentation of this paper will lead to the adoption, throughout the country, of a uniform set of standard specifications covering all systems of building construction. Mr. Post.

In view of the extremely short time usually allowed an engineer for designing the structural portion of a building, any general information which can be embodied in the specifications, or any clause which will lessen as much as possible the amount of calculation involved, will contribute to save valuable time. In the matter of dead loads it seems as if, considering the large number of well-known systems of floor construction, they might be divided into groups or classes with an approximate dead-load value for each class, as, for example, flat-tile arches, segment arches, concrete slab construction, etc., to include in each case the weights of all the material to make the finished floor.

The weight of partitions often forms a very large proportion of

Mr. Post. the dead load, and, as frequently happens, their location is materially changed after the structure is completed. Under such circumstances it would seem advisable to provide in the calculations, not only for the partitions as they are shown on the plans, but also for every other possible location, or else make suitable provision in the assumption of the live load.

The live-load units, as specified in most of the building laws, seem to be excessive, but, in the writer's opinion, the dead-load units for floors are often guessed at, and the partition weights neglected altogether, so that the result given by the combined loads is not excessive.

The following live-load units are suggested:

For apartments, dormitories, dwellings, hospitals, hotels, etc., 40 lb. per sq. ft. or 2 000 lb. concentrated at any point.

For schools, theater galleries, and churches, 60 lb. per sq. ft.

For office buildings, above the ground floor, 60 lb. per sq. ft., or 5 000 lb. concentrated at any point.

For ground-floor offices, stores, light manufacturing, stables and carriage-houses, 80 lb. per sq. ft., or 5 000 lb. concentrated.

For assembly rooms, main floors of theaters, armories, and their corridors, or for any room likely to be used for drilling or dancing, 100 lb. per sq. ft.

For sidewalks in front of stores or warehouses, it is not uncommon to see large quantities of merchandise piled up, or heavy machinery carried over, so that a load of 250 to 300 lb. per sq. ft., or a concentrated load of from 8 000 to 10 000 lb., does not seem excessive, but, for sidewalks in front of dwellings, a much lighter load might be specified.

For lofts, storage, printing houses, or for heavy manufacturing purposes the live load should be determined by the requirements of the business.

As to the bearing of beams or girders on walls, it is suggested that for convenience the area of the bearing be made to bear some relation to the size of the beam used. Referring to the tables of the strength of beams in the mill handbooks, and taking the maximum safe loads for the shortest spans given, the end reactions are such that, if the area of the template required is equal to the square of the depth of the beam, the pressure will not exceed 250 lb. per sq. in. (except possibly in the heaviest sections of 12 and 15-in. beams, which are rarely used). As the beams are usually built into a solid wall at comparatively long intervals it would seem that the pressure of 250 lb. per sq. in. would be well within the limit of safety.

A length of bearing on the template of two-thirds of the depth of the beam would be ample.

Would it not be well to embody in the specification one or more

clauses relating to furnishing for record the data upon which calculations are based, with such diagrams or stress sheets as may be necessary? It frequently happens, where alterations are made to existing structures, that such information is necessary, and is rarely to be found. Mr. Post.

GUNVALD AUS, M. AM. SOC. C. E. (by letter).—This paper has given the writer great pleasure, as it recognizes many of the objections to the common practice of designing in accordance with the empirical rules laid down by the different building codes. Mr. Aus.

The question of materials to be used and loads to be supported was discussed in the Brooklyn Chapter of the American Institute of Architects in February, 1904, and a paper, presented by the writer on that occasion, was published in *Engineering News* of April 14th, 1904. Examination of that paper will show the writer's opinion on most of the questions discussed by Mr. Schneider, so that it is unnecessary to discuss them in detail now and give reasons for that opinion.

The writer thinks that all engineers should agree to the general propositions advanced in the paper above referred to, and Mr. Schneider's admirable specification, namely:

First.—That floor beams should be designed both for a uniform load and for a concentrated load, to prevent the use of very light beams of short spans.

Second.—That floor girders—that is to say, floor members—which carry a considerable floor area should be designed for smaller live load than that for which the floor beams are designed, both because the entire area of the floor carried by such girders will never be fully loaded, and also because the loading on such girders accumulates so slowly as to do away entirely with the effect of impact to which the individual beams will always be subject.

Third.—That the columns in the lower stories should be designed for a gradually decreasing live load, as it is not within reason that all the girders supported on these columns will receive the maximum loading at the same time.

Fourth.—That the foundations should be designed for only a part of the live load coming on the basement columns, as otherwise unequal settlements will occur.

Fifth.—That the framework of a skeleton building should be so designed that it can resist wind pressure.

Sixth.—That preferably no cast iron shall be used in columns or lintels, but that cast-iron columns in no case shall be used in buildings more than four or five stories in height, and, when so used, that the ratio between the length and diameter of the columns shall be very much smaller, and the permissible unit stress also very much smaller, than is now allowed, for instance by the New York Building Code.

Mr. Aus. It is not easy to state just how big should be the uniform load and the concentrated load for which the beams should be designed, and the writer thinks the opinions of many engineers of experience should be heard before establishing these loads.

Would it not be advisable for the American Society of Civil Engineers to appoint a Committee to examine this question and make a recommendation, which in all probability would have great influence when there will again be a chance to modify the present Building Codes?

The writer is of the opinion that the uniform loads specified in Mr. Schneider's specification are ample, but he believes that the concentrated load and the load per linear foot are too great.

Safes weighing 5 000 lb. are used so rarely in ordinary offices that it would seem unreasonable to design every part of a building strong enough to support such a large concentrated load, and it is undoubtedly true that safes weighing 2 000 lb. cannot be found in 1% of the residences erected. Therefore, it seems to be unreasonable to design all residences for this excessive loading at an enormously increased cost, the more so as such a provision will tend to retard the movement toward fire-proof dwelling-houses.

The writer believes that the 2 000 lb. concentration suggested by him for offices and the 1 200 lb. suggested for residences would be ample. Special permission should be obtained from the Building Department in those few cases where heavier concentrations are to be supported.

It also appears that Mr. Schneider's specification, if adopted by the Building Department, would induce the designer, for the sake of economy, to use long spans, which, unless the design was carried out by experienced engineers (which is not always done) would tend to weaken the building materially. The typical office building or apartment-house erected to-day has beams of from 15 to 16 ft. span, spaced about 5 ft. on centers, which, under the author's specification, would call for a uniform live load of 133 lb. per sq. ft.; whereas, if the spans were increased to 30 ft., the live load would only be 66 lb. per sq. ft. Further, if girders of 30 ft. span, spaced 15 ft. on centers, supported beams 5 ft. apart, these girders would only be designed for half the live load for which the beams are designed, which appears to be too great a reduction.

The writer has for many years, as Chief Engineer of the Supervising Architect's office, in Washington, D. C., reduced the live load on the girders to two-thirds of that for which the beams were designed, and this appears to be as far as one should go in this reduction.

There is no objection to any other features of the author's specification, except that the advantage of changing the specification for struct-

ural steel from that now commonly used is not quite evident; that is, from 60 000 to 70 000 lb. ultimate strength, to that specified by the author; that is, from 55 000 to 65 000 lb. Practically all the steel which the writer has had inspected for years past, under the former limits, has had an ultimate strength of from 60 000 to 65 000 lb., so that it is not thought that a different material would be obtained under the author's specification; and, even if the material should go as high as 70 000 lb., which is very rarely the case, it can be punched and sheared readily without affecting the strength of the finished members. Mr. Aus.

J. K. Freitag, Assoc. M. Am. Soc. C. E. (by letter).—In view of the Hotel Darlington disaster and similar loose methods of building design and construction, the author is to be complimented on his timely presentation of this very important subject, and it is to be hoped that this paper may serve the purpose of leading to a general revision of the building laws of large cities, with especial reference to uniformity and modern practice. The same wide divergency in building regulations in the more prominent American cities has been pointed out and discussed at some length by the writer.* Mr. Freitag.

As regards the floor loads suggested by Mr. Schneider, the writer heartily agrees with the proposed live load of 40 lb. per sq. ft., for ordinary cases. The small live loads which have been found by such experiments as those conducted by Mr. E. C. Shankland and by Messrs. Blackall and Everett, ranging from 12 to 16 or 17 lb. per sq. ft., have tempted in some cases the use of unit loads as low as 20 lb. per sq. ft., but such recommendations should certainly be questioned and even heartily condemned in conservative practice. While 20 lb. per sq. ft. may be sufficient for average present loads in office buildings, etc., it is to be remembered that the use of an average is always dangerous, while provision should be made properly, but not extravagantly, for all possibilities of excess, either present or future. The character of a building's contents or usage is subject to extreme change. The entire building, or possibly only portions thereof, may be devoted to very different uses from those primarily assumed, so that, in spite of the provisions in building ordinances against radical change in the character or degree of floor loads, it is often difficult to balance present economy against possible maximum requirements or future possibilities. Here, as well as in the strength of materials, a sufficient factor of safety should always be applied.

The author proposes to calculate all floor beams for a concentrated load of 5 000 lb. at any point, this being the approximate weight of the heaviest portable safe which would commonly be used in offices, etc. A safe of this weight would be approximately 4 ft. wide by 3 ft. deep. The factor of safety assumed in the calculations of the floor

* "Architectural Engineering," edition of 1901.

Mr. Freitag. beams, as recommended in Mr. Schneider's specifications, would be about 4. Professor Rankine* gives a factor of safety of 4, which would be applicable to a reliable steel-frame structure, while for masonry a factor of safety of 8 is recommended under live loads. Again, on page 361, the same authority states as follows:

"The factor of safety in structures of stone should not be less than 8, in order to provide for variations in the strength of the material, as well as for other contingencies. In some structures which have stood it is less; but there can be no doubt but these err on the side of boldness."

All present types of fire-proof floor arches would come under the classification of the poorest kind of masonry, so that, if the floor beams were calculated for a concentrated load of 5 000 lb., consistency would require an ultimate arch capacity of 40 000 lb. applied over any bearing area of 12 sq. ft., or 3 333 lb. per sq. ft. Many, if not most, forms of fire-proof floor arches now in general use would fail to develop this strength. In the Denver tests, the best end-construction arch sustained a final load of only 15 145 lb., or 1 670 lb. per sq. ft. over a loaded area of 9 sq. ft.; while the best side-construction arch carried only 8 574 lb., or 953 lb. per sq. ft. over a loaded area of 9 sq. ft. These were 10-in. terra-cotta arches of 5-ft. span, hence they were fair samples of modern use. The tests made by George Hill, M. Am. Soc. C. E.,† on Melan and terra-cotta arches, by means of a self-registering hydraulic machine, show only one terra-cotta arch out of eleven tests which would fulfil the necessary requirements, *viz.*, a 10-in. hard terra-cotta end-construction arch, which sustained a total load of 57 500 lb. over a loaded area of 20 sq. ft. out of a total arch area of 22.6 sq. ft. The other ten arches ranged from 10 000 to 16 000 lb., ultimate loads. All the Melan concrete arches showed an ultimate capacity of more than the required 40 000 lb. Most, if not all, of the New York Building Department tests would also have failed to show this ultimate capacity, which would be necessary, not only under normal conditions, but under fire and water tests as well. Several of the concrete arched forms now in common use will probably exceed this required strength, but if the floor construction as a whole were to be proportioned equivalent to the concentrated load assumed for the beams, this might preclude the use of practically all flat terra-cotta arches and all slab concrete construction.

The enumeration of dead loads includes partitions, while Paragraph 2 of the Specifications for Design state that the calculations of dead loads are to be based on the weights of different materials given in Table 1. It would seem as though this table could properly be extended to give the weights of terra-cotta and mackolite partition

*"Civil Engineering," page 222.

†"Tests of Fire-proof Flooring Material," *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 542.

Mr. Freitag.

material for different thicknesses, as well as some general data regarding the weights of terra-cotta arch blocks of different materials and for different arch depths. Partitions are often classed as live loads, and in previous Chicago practice these were often assumed to be live loads distributed uniformly over the floor area at 25 lb. per sq. ft. This was on the assumption that the position of partitions might frequently be changed to suit the subdivision of office areas as required by tenants. The present New York and Chicago Building Laws, however, both require partitions to be considered as dead loads.

As pointed out by the author, the live loads on warehouse and factory floors will vary greatly, these often being far greater than might be expected, and often much less. Mr. W. L. B. Jenney, the well-known Chicago architect, had occasion to estimate the loads in the wholesale warehouse of Marshall Field & Co., in Chicago, and the low average of 50 lb. per sq. ft. was found for the total floor area, including all passageways, in spite of the very large quantities of merchandise usually stored there. The maximum load on limited areas was found to be 57 lb.

In the proposed specifications, cast iron is "practically ruled out" as unreliable and unadaptable material. This elimination of cast-iron members would be warranted if all building laws which now permit its use could be changed so as either to eliminate cast-iron columns altogether or to require the calculations of columns by reliable formulas, in which latter case there would be found to be slight, if any, economy in using cast iron; but, as long as present building laws exist, allowing the use of cast-iron columns, the writer can see no serious objection to using them under approved methods of calculations, for buildings of medium height and considerable area where wind bracing is not required beyond the stability of enclosing and partition walls. There is now under process of construction a large department store in Boston, approximately 200 by 250 ft. in area, seven stories high, besides the basement and sub-basement, with masonry walls and cross-walls. In such a case, the writer can see no objection to the use of cast-iron columns, if proportioned and designed properly.

It would also seem that Paragraph 12, relating to wind pressure, should be modified so as to limit the necessity of caring for wind pressure to those structures which would require metallic bracing. Inasmuch as the specifications refer to the structural steelwork of buildings, it would appear as though Paragraph 12 were intended to require a wind pressure of 30 lb. per sq. ft. to be calculated for all buildings having either complete or partial steel frames. This provision would not be necessary for buildings provided with exterior walls of masonry and with masonry partitions or cross-walls, especially if the base is a large proportion of, or equal to, the height.

Paragraph 54, regarding column splices, might be extended to require the breaking joints of columns alternately at any floor level.

Mr. Freitag. Paragraph 57 of Part II, regarding field painting, should be amended to require the field painting to be of a different color from the shop coat.

The intimate relation which exists between the steel frame of a modern building and the fire-proof floors and coverings makes it indispensable that the successful designer of the steelwork be also thoroughly familiar with approved fire-proofing methods, and successful work along fire-proof buildings would require, not only such careful specifications of the steel frame as Mr. Schneider has prepared, but also as careful specifications relating to the fire-proofing. The question of floor girders could be taken as an instance. These are often designed without reference to partitions and without reference to flush, unbroken ceilings. The experience gained through the fire in the Horne Buildings in Pittsburg, and elsewhere, has shown that far better results as regards the fire-proofing are secured from flush ceilings than from paneled ceilings where the girders are allowed to project below the ceiling line. Again, the successful design of spandrel beams or members can only be accomplished by carefully considering the fire-proofing possibilities of the spandrel construction. The successful designer of steel-frame buildings, therefore, should be as familiar as possible with the whole range of fire-proofing, and also as familiar as possible with the architect's standpoint and the problems which he must face.

Mr. Hewes. VIRGIL H. HEWES, M. AM. SOC. C. E.—The different changes of loading which take place after buildings have been erected, and the impossibility of predicting what the changes will be, having been mentioned, the speaker would like to cite a case, which came under his observation while making an examination of a building in New York City upon which the Building Department had filed a violation.

The plan of the building was of flat-iron type, being wider at one end than at the other, with two sides converging. The walls on one side and on the larger end were supported upon cast-iron columns at the street or sidewalk level; at the smaller end the wall ran down to the foundation. The other side was broken by re-entrant walls to form a light shaft. These walls were also supported on cast-iron columns, while the two sections of wall along the lot line extended down to the foundation.

All the tenants had moved out of the building, except those in one store at the street level, and a printing office on the top floor. The printing office was being moved; everything had been taken out except a large press which was still in place. The pressman, having a piece of work which he wished to turn out before taking down the press, started it up at the highest speed. The table carrying the forms, having a reciprocating motion, caused the building to vibrate, increasing up to a point where it took a gyrating motion, then the motion would

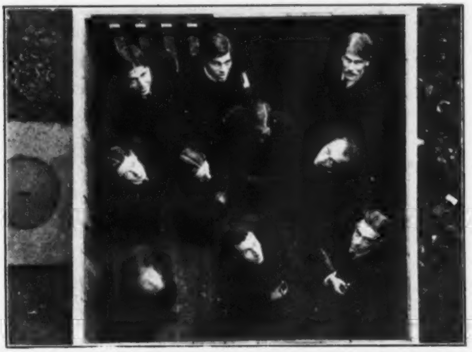


FIG. 1.—CROWD WEIGHING 1 505.8 LB., TOTAL; OR 41.8 LB. PER SQ. FT.

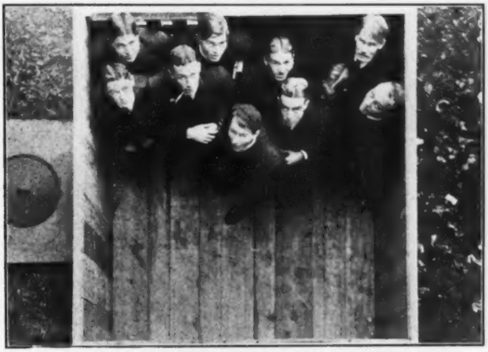


FIG. 2.—SAME CROWD AS IN FIG. 1, BUT LESS SCATTERED.

FIG. 3.—CROWD WEIGHING 3 013.4 LB., TOTAL; OR 83.7 LB. PER SQ. FT.

die out and then start to vibrate again to the point of maximum vibration. Mr. Hewes.

The speaker was two floors below the printing office when the press was started, and notified the men of the chances they were taking. then he took a vacation till the press was stopped. It is probable that if the lower floors had been loaded, the vibrations would not have been as severe.

This tends to show how necessary it is that the frame of a building should be a structure in itself without depending upon the partitions and walls for stiffness, as Mr. Schneider has stated.

L. J. Johnson, M. Am. Soc. C. E. (by letter).—Mr. Schneider has certainly done the Profession a great service in publishing this paper. The writer begs to express his share of the gratitude for it. While so doing, he wishes to discuss one point which Mr. Schneider brings up —the weight of a crowd of people. He is not reconciled to such low figures as Mr. Schneider cites for this quantity. By his recommendations and his assertion, that "a uniform load of 40 lb. per sq. ft. will scarcely ever be exceeded by a crowd of people," he may give the impression that 40 lb. per sq. ft. is a fair estimate of the actual weight of a closely packed crowd, an impression which will not be found to be substantiated by facts. Mr. Johnson.

A few months ago the writer made some experiments on the weights of crowds of his students, and found that 156 lb. per sq. ft. was attainable without any attempt at selecting the men or crowding them to any painful degree of personal discomfort. Results nearly as high are reported by Mr. Stoney, by Professor Kernot, of Victoria, and by C. M. Spofford, Assoc. M. Am. Soc. C. E.*

This knowledge is often important in design. Allowance for it, of course, may be made in any way that may be approved by the judgment of the designer, but, in any event, it should be understood clearly that 150 lb. per sq. ft. may be reached, or even exceeded, under a crowd of people at points subject to special congestion.

For example, students among those who formed the crowd above mentioned expressed the conviction that the throngs leaving the university football field after large games are at one place compressed quite as tightly as were the students in this test. This special congestion occurs upon a drawbridge the width of which between railings is considerably narrower than the street which leads to it. The writer has vivid recollections of being in crowds compacted by the bridges at the Columbian Exposition on Chicago Day, where the density of the crowd must have been quite as high as that of the crowd of students when weighing 156 lb. per sq. ft.

While 150 lb. per sq. ft. may be reached only rarely, 80 or 100 lb. per sq. ft. must be realized far more often than is commonly supposed.

* *Engineering News*, Vol. LI, pp. 360 and 496.

Mr. Johnson. A crowd of 80 lb. per sq. ft. can easily make way so as not to afford serious obstruction to the progress of a person who wishes to go through it, and a little persistence will enable a person to make his way through a willing crowd weighing 120 lb. per sq. ft. The details of the experiments on which these assertions are based, and a collection of citations from American and foreign authorities will be found in *Engineering News.**

It has occurred to the writer that photographs, showing just what degree of congestion is indicated by loads of about 40, 80, 100 and 150 lbs. per sq. ft., would be of interest at this point. Consequently, a number of volunteers from among the writer's students were weighed and caused to stand in a box made for the purpose. This box was 6 ft. square, in the clear, inside measurement, and with vertical walls, 5 ft. 9 in. high, and without a top. The men filed into the box, and photographs were taken as the weights reached the requisite totals. The camera was at a window some 20 ft. above the top of the box, and the men were asked to look up, so as to be more readily identified and counted, as a check upon the record. The results are shown on Plates XL and XLI.

Figs. 1 and 2, Plate XL, show the same group of students. In Fig. 1 they are distributed over the available area, and in Fig. 2 they are assembled along one side of it. These ten men aggregated in weight 1 505.8 lb., which, on the 36 sq. ft., made a load of 41.8 lb. per sq. ft.

Fig. 3, Plate XL, shows the same men and ten additional men, bringing the total up to 3 013.4 lb., and unit load to 83.7 lb. per sq. ft.

In Fig. 1, Plate XL, four additional men bring the figures up to 3 601.7 lb. total and 100.0 lb. per sq. ft.

In Fig. 2, Plate XLI, thirteen additional men, making thirty-seven in all, bring the results up to 5 552.5 lb. total and 154.2 lb. per sq. ft. The average weight of these thirty-seven men is seen to be 150.1 lb.

In Fig. 2, Plate XLI, no attempt was made to reach a maximum, but only a full 150 lb. per sq. ft., and a number of the men testified that the congestion seemed to be materially less than that to which they are subjected upon the drawbridge above referred to. Obviously, there are several very short men in the picture who could be replaced by taller men occupying little or no more room, and it seems to be clear that 160 lb. per sq. ft. is quite within the possibilities.

Fig. 3, Plate XLI, is another view of a crowd under which the average floor load is 41 8 lb. per sq. ft. The five men shown are in an alcove, 4 ft. square, and their combined weight is 669 lb.

In the light of these experiments, the writer is convinced that 80 and 100 lb. per sq. ft. are of common occurrence throughout whole aisles and passageways, and even 125 lb. cannot be infrequent. The writer knows of grand stands where 3.3 sq. ft. is the allowance per

* *Engineering News*, Vol. LI, p. 360.

FIG. 1.—CROWD WEIGHING 3 601.7 LB., TOTAL; OR 100.0 LB. PER SQ. FT.

FIG. 2.—CROWD WEIGHING 5 552.5 LB., TOTAL; OR 154.2 LB. PER SQ. FT.

FIG. 3.—41.8 LB. PER SQ. FT. IN ALCOVE.

person seated. This, assuming 150 lb. as the average weight per person, would make 45 lb. per sq. ft., with no allowance for the weight of the seats themselves. Mr. Johnson.

It is freely admitted that the writer's results give figures greatly in excess of those given by the accepted authorities (outside of some municipal building laws), both in the United States and in Europe, but the experiment is one very easily tried by anyone who may feel unconvinced.

Doubtless, mixed crowds of men and women, such as football spectators, may weigh less per square foot, with an equal degree of personal discomfort, than the body of students in the writer's experiments.

It should be remembered that a closely packed crowd is not likely to be in a mood to take calmly any undue deflection or appearance of weakness in the floor, and the result of such seeming insecurity is not pleasant to contemplate. In the writer's opinion, such floors as those of passageways, corridors, standing-room in theaters, assembly rooms without fixed seats, ballrooms, etc., should be calculated for a weight closely approaching 150 lb. per sq. ft., or, in some cases, even more, without exceeding the unit stresses of Mr. Schneider's Paragraph 17. Possibly, a large standing assemblage, such as is common at political meetings, likely to applaud by stamping; or, a throng of dancers; or a body of drilling solders, might call for an additional impact provision. Moreover, it should not be forgotton that in an assembly room "with fixed seats" those seats are sometimes removed in order to accommodate as many as can be packed into it standing.

To summarize briefly, the writer begs to maintain:

I.—That the extreme value of the statical load from a crowd of men is a very few pounds, if any, below 160 lb. per sq. ft.;

II—That there are many structures which contain considerable areas where a load as great as 150 lb. per sq. ft. is to be expected occasionally and fully provided for;

III.—That these facts should be clearly stated, and that the maximum loads should not be left to be taken care of by a concentrated-load specification which might or might not provide for them according to the closeness of the beam spacing;

IV.—That the distributed-load values given in Paragraph 3 ought. accordingly, to be increased materially, at least for ground floors of office buildings, assembly rooms, and staircases leading thereto, and in many cases for sidewalks.

As a supplement* to his previous discussion, the writer begs to present some additional data upon the weight of a crowd of people, the gist of which is to be seen in the photographs on Plates XLII

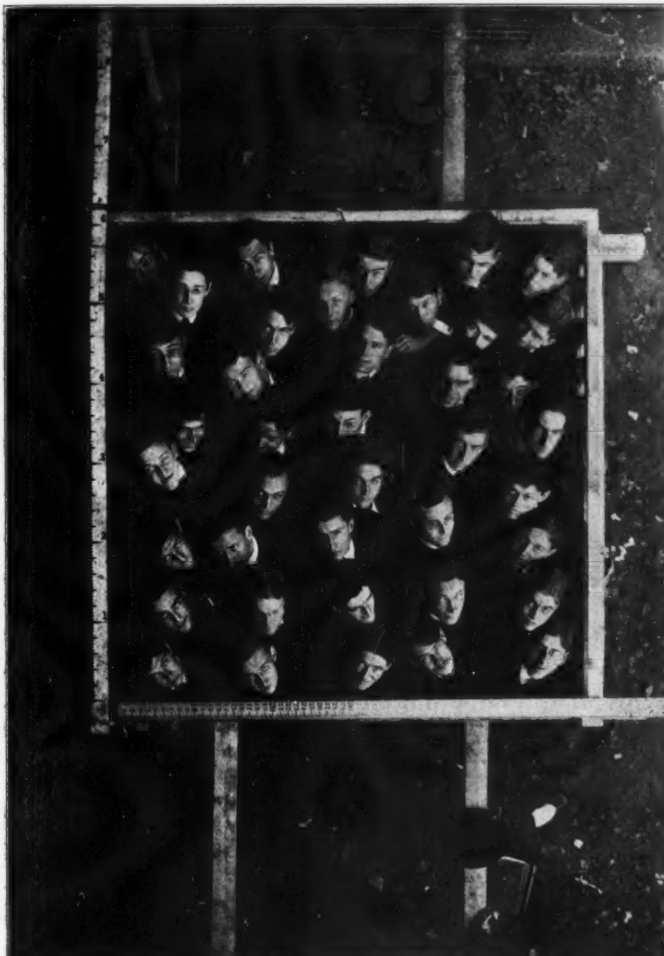
* This part of Mr. Johnson's discussion contributed subsequently.

Mr. Johnson. and XLIII, the most important of which is Plate XLII. These later results show that even the 160 lb. per sq. ft., mentioned previously, may be considerably exceeded.

In all the writer's previous experiments the men took their places in the enclosure entirely at random, with a maximum result of 156 lb. per sq. ft. Since, however, in crossing a bridge or in a packed assembly hall, all the people face one way, the experiments were continued to see how much a crowd of this kind would weigh. The same box (6 by 6 ft., inside measurement) as in Plates XL and XLI was used, and the men were selected, somewhat, seeking on the whole tall and preferably slender men. A result of 176.4 lb. per sq. ft. followed the first trial of this kind. Forty men, averaging 158.8 lb., entered the box, and the gate (in one of the sides) was shut and barred. Study of the resulting photograph showed that the maximum had not been reached, and the experiment was repeated with more care as to the selection of the men. The result was (Plate XLII) 181.3 lb. per sq. ft. from forty men averaging 163.2 lb. each. The men, all undergraduate students of engineering, ranged in weight from 119.6 to 203.1 lb., twelve of them weighing less than 150, and ten more than 175 lb. The stadia rods in the photograph furnish a means of verifying the size of the box, and the men may readily be counted. The figure, 181.3 lb. per sq. ft., may be looked upon as very close to the upper limit of the weight of a crowd of people, but a competent and careful observer of the test resulting in 176.4 lb. per sq. ft. declared that in his opinion the congestion did not differ much from that of the crowd on the drawbridge after foot-ball games, referred to previously, and this is borne out by the testimony of the men themselves. It will be freely admitted that if forty men, averaging 163 lb., can be made to stand in 36 sq. ft., forty men of the average size, 150 lb., could be placed there with comparative ease. Yet forty men of 150 lb. each would lead to 166.7 lb. per sq. ft. Further, if the five men in the rear row were to leave the box, the load would still be 166.0 lb. per sq. ft.

The pressure on the side walls of the box was not noticeably great, that upon the front and back walls was undoubtedly considerable, and the bracing shown in the cuts was provided accordingly. In making this experiment, the writer was inclined to be content with thirty-nine of these specially large men in the box, but those inside vociferously declared there was room for another, and it proved to be true.

It seemed worth while to procure for record a view of a crowd at about 125 lb. per sq. ft., and, accordingly, a picture (Fig. 1, Plate XLIII) was taken after twelve of the men in Plate XLII had left the box. Those who left were among the lightest of the men, and the average of the remaining twenty-eight rose to 167.7 lb. The weight was 130.4 lb. per sq. ft.



CROWD WEIGHING 181.3 LB. PER SQ. FT.
40 MEN, AVERAGE WEIGHT OF EACH, 163.2 LB., IN SPACE 6 FT. SQUARE.

Mr. Johnson.

Mr. Cooper's reference, earlier in this discussion, to 45 lb. per sq. ft. as the weight of the most densely packed crowds on the New York Elevated trains adds special interest to that figure. Accordingly, Fig. 2, Plate XLIII, was added, showing 47.2 lb. per sq. ft. It is due to eleven men, averaging 154.6 lb., standing in the same 36 sq. ft. as in the previous views. It does not accord at all with the writer's recollection of New York Elevated trains during rush hours.

It may be interesting to add that what may be called the asymptotic value of the weight of a crowd of men must be about 218 lb. per sq. ft. (possibly more than this rather than less, with men of varying height). This figure was reached upon examination of data kindly furnished by Dr. Sargent, Director of the Harvard Gymnasium. It was obtained by dividing the weight of a man, 6 ft. 3 in. tall, a former foot-ball captain, by his maximum horizontal cross-section as obtained by a planimeter. This maximum section, of course, was through the chest, including the arms. The weight of this man was 177 lb., and the maximum cross-section was 117 sq. in., both quantities being exclusive of clothing.

With these photographs before him, the writer sees no escape from the conviction that crowds of 167 lb. per sq. ft. (forty average men in 36 sq. ft.) are entirely likely, and that from 130 to 140 lb. must be commonly reached in all places where crowds of people congregate standing. The careful designer, moreover, will remember that 180 lb. per sq. ft. is within the range of probabilities.

The margin of safety in many existing structures designed for from 80 to 100 lb. per sq. ft. (to say nothing of 40 to 45) must be much less than has been supposed. Probably the correct inference is that the experience of many years in many lands has demonstrated that the margin has been sufficient, nevertheless. Even if that be true, it is no reason why engineers should remain in the dark about how much a crowd of people actually weighs.

Perhaps the logical course to pursue now would be to increase the allowable working stresses so as to compensate to some extent for the increased loads actually assumed. Perhaps, on the other hand, the present unit stresses may properly be retained and more care be devoted to deciding for what portion of the possible maximum load special structures or parts of structures may properly be designed. This latter course seems the more scientific to the writer, but the subject thus opened is one which is always a fair field for the play of individual professional judgment, and into which the writer does not care to proceed farther at present. He begs to say, however, that he deplores the tendency, apparent in some quarters, to overlook the fact that a crowd of people is the very last load to be endangered by too low a margin of safety, even "once in a great while."

Mr. Johnson. The writer begs to record his indebtedness to E. E. Pettee, Assoc. M. Am. Soc. C. E., and to N. E. Olds, one of his students, who took the photographs accompanying this discussion.

Mr. Macdonald. H. P. Macdonald, Jun. Am. Soc. C. E.—The speaker wishes to take exception to Mr. Llewellyn's remarks about the inspection of castings. The obstacles to the inspection of castings when they first leave the sand are more those of the inspector's than the foundryman s making. The average inspector does not care to get around at 4 or 5 o'clock in the morning when the pieces cast the day before are shaken out, and, as the foundryman has to use his flasks, he cannot wait until the inspector eats a late breakfast.

The speaker does not think that any one of the big foundries around New York City would raise the least objection to an intelligent inspector watching the manufacture of its product from the time the metal is charged in the cupola until it leaves the machine shop, but would rather welcome his advice and suggestions. An inspector who can be deceived, by the use of "Smooth On" or such compounds, into passing a defective casting, does not know his business, or is careless in his work.

A case of very excessive floor loading in an office building has recently been brought to the speaker's attention. In a room, 13 by 16 ft., were stored 1 500 000 pamphlets, weighing 25 lb. per thousand, besides a 1 600-lb. safe and sundry articles, which brought the total load to more than 40 000 lb., giving at least 200 lb. per sq. ft. of floor area.

Mr. Goodrich. E. P. Goodrich, Jun. Am. Soc. C. E. (by letter).—Mr. Schneider's restriction of his subject to buildings is important in connection with the formation of a specification for the design of pile foundations, as well as for that of other parts. It is inferred, as a further restriction, that his intention is to treat only structures in which a steel frame is to be found, so that the building may be expected to be a rather large one and the foundation loads of some size. Where this is the case the foundations are massive, and the effect of vibration may be considered *nil*, except perhaps in some machine shops and buildings of similar nature.

Even with pile foundations, unless the piles are driven to rock or other firm bearing stratum, it is important to proportion the areas of the foundations or the number of piles used in each, so that exactly equal settlement will be secured at all points. In order to do this, a proper relation must be established between the live and dead loads, proper values must be assumed for the bearing powers of various soils, and a correct pile spacing and supporting power must be assumed for the kind of earth and for the other factors entering the problem. Of course, none of these items can be determined beforehand with any great degree of accuracy or even that which approaches the degree of certainty now attained in steel design, and every means

FIG. 1.—130.4 LB. PER SQ. FT.
28 MEN. AVERAGE WEIGHT 167.7 LB.

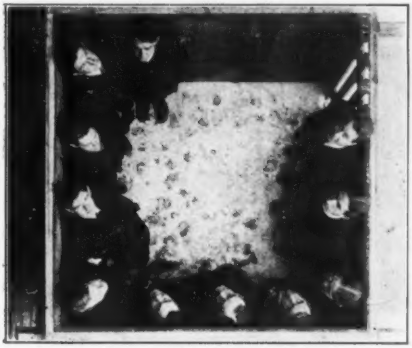


FIG. 2.—47.2 LB. PER SQ. FT.
11 MEN. AVERAGE WEIGHT 154.6 LB.

which is likely to diminish the possible or probable error should therefore be used. Mr. Goodrich.

The writer considers the method given by Mr. Schneider for the determination of a unit strain on foundations as perhaps the best that can be devised, except that it will be better to take, not a column with the greatest relative live load, but one with an average live load. In this way the percentage of possible variation would be cut in two. In the case of power plants, etc., it will often be better to build so broadly that practically no settlement will occur in any case, and to provide entirely separate foundations for all parts likely to be subjected to excessive loading. In warehouses, little or no reduction of load can be permitted.

The values assigned by Mr. Schneider for the permissible pressures on various soils are perhaps as close as may safely be determined empirically, but the values given seem to the writer needlessly conservative. Recourse should always be had to direct experiment wherever possible, and it is well to note that some settlement must always be allowed for; and that it is infinitely better to test a relatively large tract (from 4 to 6 sq. ft.) with the actual load expected, than to overload greatly a tract of only 1 or 2 sq. ft. Furthermore, this test should be made as near the actual base of the building foundations as possible, not at the surface of the ground, and the test should be made with filling packed around the bearing mass and not in an open hole. These methods may seem to be unnecessarily exact, but their value has been proven in the writer's experience.

The writer thinks Mr. Schneider's paragraph on "The Bearing Power of Piles" can be somewhat improved. It is understood that Mr. Schneider intends to omit from his specification all that part relating solely to construction work, except in so far as the word "workmanship" covers it. With this point in mind, the first objection is, that the design of the foundations and their construction have been confused. A designer must assume a certain unit bearing power, and dimension his foundation accordingly. The superintendent of construction has to see that the unit stresses assumed by the designer are actually to be provided for, whether they be for concrete, earth or piles. Therefore, it would seem that the including of the pile formula in a specification for unit stresses was putting it in the wrong place. The first two sentences of the paragraph in question are permissible, except that it would be better to specify that piles are to be computed as round-end columns whenever they are to be driven to rock or an equivalent bearing stratum, and as columns fixed at one end and of a proper length when driven under other circumstances. In sand, when the pile is entirely buried, 10 ft. is ample for such "proper" length, and even in the mud on the bed of a river piles have been known to fail by breaking off at the mud surface when driven only 15 or 20 ft. into it. It is usually unnecessary to state that the bearing

Mr. Goodrich. power of the pile itself is not to be exceeded, as column formulas always include that factor; and the usual stipulation as to a maximum allowable load per pile always precludes the possibility of its being even approached. The allowable end bearing assigned by Mr. Schneider for yellow pine is 1 500 lb. per sq. in. His maximum allowable load on a pile is 40 000 lb. A short yellow pine cylinder less than 6 in. in diameter would support this load, and 6 in. is the usual minimum size for the points of wooden piles.

The writer believes that the best practice is to assume a given load per pile, design all footing accordingly, and make the superintendent of construction provide and drive piles which will sustain this assumed load. In that case the designer's care will be to provide just the proper number under each footing and to space them so that each will develop its full proportion of the given load. To this end, groups should be made as nearly circular as possible, especially when they consist of any considerable number of piles. The corner piles of square groups of sixteen piles might just about as well be omitted. It is of the utmost importance not to space piles too closely together, or, if close spacing is necessary, to drive all to such depth that the bearing power of the earth at that depth is sufficient to provide the necessary supporting power. All the piles under a building should be driven to the same depth, if possible, and the areas of groups should be carefully proportioned to the loads carried, unless the spacing is great enough for each pile to develop its full supporting power independently. Tests* made by the Department of Docks and Ferries, of New York City, prove conclusively that piles driven in the North River mud, even to considerable depths, influence each other to some extent when 6 ft. apart, and are practically a unit in their action when only 3 ft. apart. A group of two piles thus spaced had a supporting power of only about one and two-thirds times what a single pile developed when properly spaced.

Under the circumstances, it seems better to devise some rule for the spacing of piles under foundations, make each pile carry an equal load, and drive all to the same depth. If the earth is uniform in character, this depth seldom need be very great, but, if it is not uniform, the piles should be driven through the variable stratum if possible. Under such circumstances, no pile formula is needed, because it is only necessary to drive until a specified penetration is attained in each case under a standard height of fall of hammer. From the writer's experiments† he believes that a penetration of 1 in., when produced by a 2 000-lb. hammer falling 15 ft., as freely as possible with rope attached, will give most satisfactory results. With hammers of different weights the same drop should be maintained and

*See "Wharves and Piers," by John A. Bensel, M. Am. Soc. C. E., Papers—International Engineering Congress, 1904, *Transactions*, Am. Soc. C. E., Vol. LIV, Part F, p. 1.

†"The Supporting Power of Piles," by E. P. Goodrich, *Transactions*, Am. Soc. C. E., Vol. XLVIII, p. 180.

Mr. Goodrich.

a corresponding penetration depended upon. Higher falls are much less effective. In some cases penetrations amounting to as much as 30 in. have proven satisfactory under conditions which will be discussed later. The experiments, and actual tests made, tend to show a supporting power of 100 000 lb. for such a pile when acting alone and free to develop the full effect of the blow producing a 1-in. penetration. Theoretically, a pile would have to be driven about 50 ft. into earth having a high ratio of lateral to vertical pressure (such as silt), in order to attain a frictional resistance of this amount, and to a depth of about 40 ft., to reach a stratum at which bearing and frictional resistances combined would develop this amount. With dryer soils and those containing much coarse and fine sand mixed, the depth, theoretically, would not be as great, it being slightly more than 30 ft. These depths are given on the assumption that the materials through which the piles are driven are practically homogeneous, and ignoring the considerable increase in supporting power which always occurs as soon as the piles are left to stand without interference even for a few hours. In the case of moist sand, this increased power often amounts to nearly double the original value, and, under certain circumstances, in the case of mud, to as high as ten times the original amount. This increase in supporting power is due to the settling back against the side of the pile of the earth disturbed in driving. This settling action is often very rapid, as evidenced by the difference in the effect of steam and gravity hammers. In fact, the writer believes the rapid blows of the quick-acting steam hammer are more than twice as efficient as the more measured ones (of equal theoretical effect) of gravity hammers. For these reasons, it is evident that a pile formula cannot be used indiscriminately, and that the actual supporting power of the soil must be taken into account in most cases.

With these ideas in mind, the maximum of 40 000 lb. proposed by Mr. Schneider seems very conservative, giving a factor of safety of 2½ with the specification proposed by the writer and according to his formula, and a factor of 4½ if tested by the *Engineering News* Formula, without the factor of safety introduced. Wellington proposed the almost universal introduction of a factor of safety of 6, and it is introduced in the formula given by Mr. Schneider. It must be remembered that all formulas properly apply to piles only at the time of driving, with the probability of the above factors of safety being doubled with lapse of time with most soils, and, in many cases, with the possibility of increasing them four-fold.

Under any circumstances, it is not necessary to drive piles harder than just enough to develop a supporting power of two and one-half times 40 000 lb., if that be specified as the proper factor of safety and maximum allowable load per pile.

Mr. Goodrich.

Fig. 5 shows the curves of maximum supporting power of the *Engineering News* (Wellington) Formula and of that of the writer, for different penetrations, on the assumption that $W H = 1$; it also gives a ready means of finding the value of $W H$, the factor of safety, or the penetration, when the other two terms are assumed, and a supporting power of 20 tons is desired.

A value of $W H$ of 15 ft-tons is about as small as it is economical to use in work of any magnitude, and a value of 30 ft-tons is far above good practice. It is thus seen that penetrations of less than $\frac{3}{8}$ in. are little used, and, in any case, the writer would absolutely exclude those less than $\frac{1}{4}$ in., and prefers to use only those amounting to 1 in. or more. This restriction, together with that as to a maximum load, reduces practically all pile formulas, for their curves, between the limits given, to almost parallel ones, so that results differ only in the factor of safety which the author purposely includes or unwittingly introduces in the making up of his formula. On this point there is almost as much difference of opinion as there is difference in formulas.

Thus, it seems altogether best to exclude all formulas from incorporation in a specification of "unit stresses" and, under "workmanship," simply to state the conditions required to afford the assumed stresses, basing the conditions on the best experimental evidence.

If it be assumed that the angle of internal friction of earth has a tangent of 0.4, the allowable spacing from center to center which will develop a bearing power of 40 000 lb. per pile with a factor of safety of $2\frac{1}{2}$ at the depths to which the pile is driven, is given by the following table:

Depth to which driven.......	10	15	20	30	40	50 ft.
Minimum spacing necessary to develop load..............	4.0	3.3	2.7	2.2	2.0	1.8 ft.

It is to be remembered, however, that it is not possible to drive piles too closely into earth, because the latter has only a limited compressibility. With the spacings given above, the theoretical actual increase in density of the earth, if 12-in. cylindrical piles be driven, is as follows:

Spacing................	4	3.3	2.7	2.5	2.2	2.0	1.8 ft.
Percentage of increase in density of earth after driving.............	5	10	12	15	21	25	33 per cent.

Earth with 35% of voids, if compressed so that all voids are filled, will increase in density only 54 per cent. From quite a number of tests of the compressibility of soils, made by the writer, it is evident that a tremendous amount of energy is wasted in pile driving if the

Mr. Goodrich.

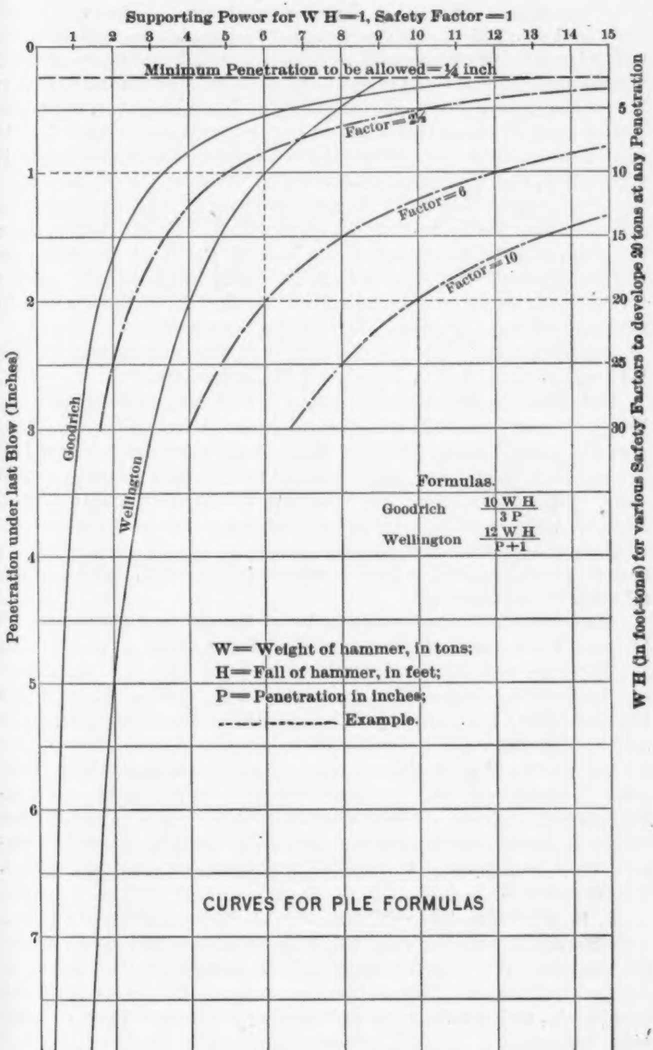


Fig. 5.

Mr. Goodrich. piles are spaced so closely that any great compressing of the soil must be done. This wasted energy is not disclosed in any pile formula, and serves to give exaggerated values when such formulas are applied. Considerable practical experience also confirms this and all the other theoretical results given above. Thus, it is evident that, even with piles spaced $2\frac{1}{2}$ ft. apart, the amount of compression suffered by the earth is more than one quarter of the maximum possible amount in many cases, and that considerable energy must be wasted in driving so closely. A spacing of 3 ft. is much to be preferred, especially when it is seen that the theoretical depths to which it is necessary to drive the piles, in order to develop a safe bearing power of 40 000 lb., are 16 ft. for the 3-ft. spacing and 26 ft. for the $2\frac{1}{2}$-ft. spacing. The writer thinks that a minimum spacing of not less than 2.7 ft. should ever be allowed and that 3 ft. should be used wherever possible.

The writer therefore proposes the two following paragraphs on "Piles," in place of the one given by Mr. Schneider.

Unit Stresses.—All pile foundations are to be designed so as to bring, as nearly as possible, a load of 40 000 lb. on each pile. Piles are to be spaced not closer than 2.7 ft. from center to center, and all groups are to be made as nearly circular in general outline as possible. Piles are to have such diameters as will afford ample stiffness and give sufficient area to act as columns, considered as pin-connected when driven to rock or equivalent bearing at depths less than 20 ft., and considered as fixed at the lower end and of 20 ft. length under all other circumstances.

Workmanship.—All piles are to be of lengths and diameters not less than those specified, are to be spaced accurately as shown upon the drawings, and driven as nearly as possible to a uniform depth, provided uniform penetration is developed under equal blows of the hammer. Such penetration is to be 1 in., as nearly as possible, under a 15-ft. blow from a 2 000-lb. gravity hammer, or a proportional penetration under any other weight of hammer with the same fall. Only piles of length necessary and just sufficient to develop this penetration are to be driven. Should a steam hammer be used, equivalent values of the hammer weight and the height of fall of a gravity hammer are to be used, and a penetration of 2 ins., as nearly as possible, is to be secured.

Mr. Ketchum. M. S. Ketchum, Assoc. M. Am. Soc. C. E. (by letter).—This is a valuable and timely paper, and it is to be hoped that, together with the discussion, it will lead to more rational methods for the structural design of buildings. The author has covered the entire field very thoroughly, and, for the most part, the specifications meet with the writer's approval.

Without attempting to discuss the specifications as a whole, the writer would call attention to the following paragraphs:

Paragraph 7.—The weights of roofs and roof coverings vary so much that it would appear to be more logical to calculate the weight of trusses, purlins, sheathing and roof covering in each case. For calculating the weight of roof trusses for mill buildings, train-sheds, etc., the following formula has been proposed by the writer: * Mr. Ketchum.

$$W = \frac{P}{45} A L \left(1 + \frac{L}{5 \sqrt{A}}\right)$$

in which W = weight of roof truss, in pounds;

P = capacity of the truss, in pounds per square foot of horizontal projection of roof;

A = distance from center to center of trusses, in feet; and

L = span of truss, in feet.

Paragraph 8.—In localities subject to snowfall, it would seem desirable to consider a minimum ice or sleet load, of say 10 lb. per sq. ft., which would be on the roof at the time of maximum wind. The author appears to have had the sleet load in mind in specifying a minimum snow load of 10 lb. per sq. ft

Paragraph 17.—The writer would think it more rational to specify the allowable shear on the net section of webs of plate girders.

Paragraph 22.—Cross-bending and direct stress should be combined by the application of a rational formula which takes account of the fact that transverse loads produce larger stresses in compression members than in tension members. The writer has used Johnson's formula,†

$$f = f_1 + f_2 = \frac{M_1 \ Y_1}{I \pm \frac{P \ l^2}{C \ E}} + \frac{P}{A},$$

for combined stresses, and believes that it is the most satisfactory formula yet proposed. The proposed reduction of the transverse bending moment does not appear to be proper, except in the cases of wind moment, moment due to weight, and moment due to eccentric loading. The writer believes that the usual method, of increasing the allowable stresses by 25% when wind is considered, and by 10% when weight and eccentric loading are considered, is to be preferred.

Paragraph 37.—The writer has noted with pleasure that the author has adopted the stresses and the straight-line formula for the design of steel struts and columns proposed by the American Railway Engineering and Maintenance-of-Way Association. Under the circumstances, however, it does not appear to be consistent to use a straight-line formula for designing steel members and a curve formula for designing timber struts and columns.

* "The Design of Mill Buildings and the Calculation of Stresses in Framed Structures," Engineering News Publishing Co., New York.

† "Modern Framed Structures."

Mr. Ketchum. The straight-line formula adopted by the Cities of Philadelphia, Buffalo and Minneapolis, and used by the writer in his specifications for "Mill Buildings,"*

$$p = C - \frac{C}{100}\frac{l}{d},$$

in which p, C, l and d are the same as used by the author, would appear to be more in keeping with the spirit of the specifications.

Paragraph 62.—This clause is a decided improvement on the usual clause specifying 1¼ in. for all sizes of rivets.

Paragraph 66.—The writer believes that one-sixtieth of the distance between rivet centers will give batten plates which are too thin, and that the usual specification of one-fortieth should be substituted.

Part II, Paragraphs 2 and 3.—The use of Bessemer steel is allowed in Paragraph 2, but it is virtually cut out by the requirements in Paragraph 3. The writer favors specifying that steel shall be made by the open-hearth process except for temporary or unimportant structures.

Mr. Blakeley. GEORGE H. BLAKELEY, M. AM. SOC. C. E.—The high authority of the author will undoubtedly commend the proposed specification to those seeking guidance in the structural design of buildings, and, also, undoubtedly, will influence greatly the design of such work. The requirement of the consideration of concentrated loads in the designing of floors is a commendable provision which deserves the attention of those who have not already given the matter the attention that the importance of the subject warrants.

The loadings proposed by the author, however, should be considered carefully before general adoption, as it is a serious question whether they do not produce an asymmetrical design, making the floor joists heavier than necessary and the girders lighter than desirable, within the proper margin of safety.

If it is proper to provide for supporting a 5 000-lb. safe, then, only in exceptional cases could the entire weight of the safe be carried on a single joist. Such a safe would occupy an area of about 3 by 5 ft., and it is proper to consider such a distribution of the load in designing. With joists of 15 ft. span and spaced 5 ft. apart, it is impossible to place such a safe in any position where it would produce a loading of a single joist in excess of that caused by a center load of 3 500 lb. As the proposed specification does not purport to be a simplification of calculation, it would be proper to specify a definite area covered by the concentrated load, instead of considering it under the impossible condition of being concentrated at a mathematical point. Such a modification would produce, in general, a reduction of the sizes of floor joists, and without impairing the adequate carrying capacity.

* "Steel Mill Buildings," Engineering News Publishing Co., New York.

On the other hand, it is questionable if the specification of a live load of 1 000 lb. per lin. ft. for girders is sufficient to cover the contingencies of loading that may occur in buildings. For example, a perfectly possible case is that of a wing of an office building with two rooms, each 16 ft. square and with the girder under the partition between the rooms. This girder supports a floor area of 256 sq. ft. It is possible that the occupant of each office may have a heavy safe, which he would not place against the door partition nor against either of the two outer walls of the offices, as it might interfere with the windows, but each would place his safe against the partition wall over the girder, in which case there would be a concentration of two safes on the girder. If these were 5 000-lb. safes, then the girder would be loaded by the safes alone equivalent to a uniform load of 1 250 lb. per lin. ft , or 25% in excess of the load for which the girder was designed, and without any further provision for carrying the 256 sq. ft. of floor area which must still be supported by the girder. Of course, with certain arrangements of the floor framing, with due regard to the area of floor space occupied by the safes, and with precise calculation, the effect of the loading produced by these safes would be very much reduced, and might be even as low as an equivalent of 750 lb. per lin. ft. of girder. But, under a possible arrangement of floor framing, these safes, with due regard to their area of floor space and with precise calculation, would produce an equivalent loading of 1 000 lb. per lin. ft., thus consuming the entire carrying capacity for which the girder was designed and without leaving any remaining provision for carrying the floor space which still must be supported by it. It is quite certain that the authorities in charge of the building would direct the safes to be placed where they would be supported by the girder, though they might direct that they be placed at the wall end or at the column end of the girder, which would materially lessen the effect of the loading. In many buildings, however, there is no intelligent supervision of these matters, and the disposition of safes is left largely to the convenience of the tenants. Mr. Blakeley.

In the case of office buildings occupied by lawyers, it is possible to have bookcases filled solid with books from floor to ceiling, and on each side of the partition over a girder, producing a load of from 400 to 450 lb. per lin. ft. of girder. Such offices are usually of fair size, and, after deducting the effect of the bookcases, there may be left in the girder a carrying capacity of less than 20 lb. per sq. ft. of the floor of the offices. Such offices at times may be fairly crowded with people, as in the case of an important hearing before a referee, and may have a floor load of at least 50 lb. per sq. ft. caused by a crowd of people at such a time.

In the case of store buildings, a live load of 1 000 lb. per lin. ft. for girders is insufficient to provide for the conditions of loading that will

Mr. Blakeley. prevail in such buildings. In the construction of store buildings, the tendency is to space the columns far apart, and 20 ft. from center to center is not unusual For such a case, with joists spaced at 5 ft. centers, according to the proposed specification, the joists would be designed for a live load equivalent to 160 lb. per sq. ft., while the girders would be designed for a load of only 50 lb. per sq. ft. On a limited area in such buildings it is not unusual to have a crowd of people equivalent to at least 80 lb. per sq. ft., especially on bargain days and during the holiday season. It is perfectly possible that on such occasions areas affecting a girder in such a building will be loaded considerably in excess of 80 lb. per sq. ft , as against the 50 lb. per sq. ft. provided for by the proposed specifications. Moreover, portions of store buildings at times partake of the character of light storage buildings, in the receiving and shipping of goods. Crockery and glassware in crates, set side by side and not piled, will produce a load of 120 lb. per sq. ft. Flannels in cases, piled 4 ft. high, produce a floor loading of 100 lb. per sq. ft Cotton prints in cases, set side by side and not piled, produce a floor load of 93 lb. per sq. ft. Woolen dress goods, in cases set side by side and not piled, produce a floor load of 84 lb. per sq. ft. Brown sugar in barrels, set side by side, produces a floor load of 113 lb. per sq. ft. These and other articles handled in store buildings will at times accumulate over certain areas and fully load the girders; therefore, in the design for such buildings, the possibilities of such loadings should be considered and the girders be designed accordingly.

It is reasonable that the live-load carrying capacity of girders should have some relation to the floor area which they are to support, but, according to the proposed specification, girders spaced at 25-ft. centers would have no more live-load carrying capacity than girders of the same span spaced but half the distance apart, or at 12½ ft. centers. According to the proposed specification, each girder would be designed for a live load of 1 000 lb. per lin. ft., notwithstanding the fact that one of the girders would be called upon to support a floor area twice as great as the other. It is to be questioned if such a design will provide for the possibilities of loading which may occur.

The concentrated-load method of designing floor framing is commendable, but the concentrated load should have a specified area over which it is distributed, and such distribution should be considered in the design of the joists and the girders It is probable that no girder should be designed for a load of less than 1 000 lb. per lin. ft., but, on the other hand, it does not seem advisable to design any joist or any girder for an office building, or for a store building, for a load less than 80 lb. per sq. ft.

The reduction of live loads on columns is in more or less general use, but it is questionable if it is proper to consider any reduction of

column loading in warehouse buildings. It is not quite clear that the proposed specification sanctions such a reduction of column loads for buildings of this type, Mr. Blakeley.

In selecting the proper live loading in the design of buildings, too much consideration should not be given to the question of probability, but due and proper regard should be given to reasonable possibilities of loading which may occur. A specification for general use should be very carefully framed in this respect, and, in the design of a building, should not leave an opening for work which might prove to be inadequate for reasonable possibilities of loading.

JOHN B. CLERMONT, ASSOC. M. AM. SOC. C. E.—It is wise, on the part of the designing engineer, in proportioning a structure, to consider that there is something more than low theoretical live loads, in designing office buildings, churches, theaters, halls and other public buildings, especially in cases where alterations may be considered as probable. Mr. Clermont.

In a certain case, alterations in an office building involved the moving of a large steel vault, which had been erected on the second story and supported on brick foundation walls to bed-rock. This vault had to be transported over the floors of a portion of the old building and a portion of a new building adjoining. Its weight was about 12 tons, and the floors of both buildings were designed for a live load of 150 lb. per sq. ft.

In another part of this building, also on the second story, some changes were made in the steel construction, increasing the sizes of beams and girders and strengthening the supporting columns, in order to support another vault, weighing about 246 tons when complete. All parts of this vault were in sections, excepting the vestibule and door, and these weighed 17 and 10 tons, respectively. These pieces had to be transported over a section of the regular framing for a distance of about 40 ft. It was again found very advantageous to have a floor construction designed to carry a live load of 150 lb. per sq. ft.

While the two foregoing cases may be considered in a measure as extremes, specific cases of overloading in public buildings are of frequent occurrence. These may be caused in various ways, such as overcrowding of persons in small spaces during fire or panic; tenants in office buildings securing storage space and loading the floors with paper, bulky sample goods, records, etc., as high as the ceilings will permit; in offices which are used for show rooms where heavy case goods are constantly handled and often stacked high for lack of space; in fact, it seems almost impossible to foresee the numerous variations in live loads possible in all kinds of public buildings.

The variations in live loads, amounting to as much as 100% in the building laws of different centers of population, mentioned in Mr.

Mr. Clermont. Schneider's paper, are easily accounted for on the ground of their vastly different requirements, but as to the conditions of their commerce and population and because each is based upon local experience. In consideration of this, the establishment of a uniform standard for live loads, in order to harmonize their constructions, would mean to do no less than harmonize their conditions, and would not be conducive to the best results. By this it is not meant that the disparities between their different requirements by law are not subject to a general system of revision for similar requirements, but that the difference between the accepted good practice of these several places and the acceptance of a uniform maximum standard live load of 40 lb. per sq. ft., would be a step in the wrong direction, and, with few exceptions, for all public buildings, a live load of less than 100 lb. per sq. ft. ought not to be considered.

Attention has been called to the shifting of cores in round cast-iron columns. In an office building erected in New York City about twelve years ago, and well inspected during erection, it was found before its completion that one of the interior columns on the first story was cracked for a distance of about 5 ft. below the shelf brackets at its top. This was a case of core shifting directly in the line of the joints of the moulds. The crack was straight down along the joint mark, and, while the column was of ample section to support the load, it was necessary to place stout wrought-iron straps around it. Recently, these straps had to be removed in order to make room for a marble covering, and, at the speaker's suggestion, the column was wound with light, flexible, seven-strand cable. This is an example of what may happen if the inspection of cast iron is not thorough.

Mr. Lowinson. Oscar Lowinson, Assoc. M. Am. Soc. C. E. (by letter).—The author's effort to establish a standard specification for structural work is deserving of the highest commendation. and its success will be demonstrated by its adoption, with such modifications as more detailed experience than the author possesses in parts of the field which he has covered, will be a tribute to him for having brought it forward as a standard of reference. The following comments and suggestions are made in reference to some of the matters specified, and the hope is expressed that the compiled results of the discussion will be used as a standard to be changed only by reason of changing conditions or increased knowledge.

In the first place, the author is warned that his live loads for dwellings, hotels and apartment-houses are too small. Take a dwelling, for instance. In the life of every family there occur periods during which the apartments are crowded. Engineers are compelled to design buildings to meet the most unfavorable conditions of loading, and must be prepared for, not only a crowd, but a crowd stamping at the same time, which causes vibration in the building and must be

provided for. The writer suggests for dwellings a minimum live load of 65 lb per sq. ft. In country residence work, it is the writer's practice to design the first floor for a live load of not less than 80 lb. per sq. ft., and in city houses the sizes of the beams and girders are usually determined by their deflection, which should not exceed $\frac{1}{30}$ in. per ft. because of the danger of cracking plastered ceilings. Mr. Lowinson.

In the case of hotels, the lobby and such rooms as may be occupied for public purposes should be placed in the same class as assembly rooms.

The author separates assembly rooms into two parts, those with fixed, and those with movable, seats. This is questionable practice, for it is frequently the custom, in New York City theaters and assembly rooms, to lay a secondary floor over the seats. The writer recommends for such buildings a loading of 125 lb. per sq. ft., his reason being that, under crowded conditions, such as during political meetings, the live load frequently amounts to 100 lb. per sq. ft., and the vibration caused by stamping may easily increase this to the equivalent of 125 lb. Further, in view of Professor Johnson's recent experiments, wherein he obtained even greater loading in crowds, the writer believes the loading adopted by the author to be too light.

Stables and Carriage-Houses.—Stables and carriage-houses should be designed for automobile loads. The writer weighed some automobiles a short time ago, and found that a carriage automobile weighed 4 000 lb., with a concentrated loading of 1 500 lb. on a wheel. The writer would design a private stable in accordance with the loading given by the author, but for the carriage-house of a stable where trucks might be stored he would assume a greater load, his New York City practice being to design such a floor for a live load of 250 lb. per sq. ft. The stalls he would design in accordance with the author's figures.

Sidewalks.—It has been the writer's practice to design sidewalks for a live load of 350 lb. per sq. ft., and he would suggest that the distributing load be made equal to that figure.

Warehouses and Factories.—The writer has frequently been called upon to determine the weights on warehouse floors, and has found loads of 350 lb. per sq. ft. and greater. He would not recommend less than 250 lb. where either paper or iron is to be stored. In fact, a storage building will frequently suffer because of this.

Though hardly pertinent to this discussion, an instance may be cited where a collapse occurred in a warehouse used to store barrels. Owing to vibration in the building, caused by passing trucks, the barrels became wedged, and threw a bearing wall out into the street. Wedging of this kind will concentrate at times an enormous load on a single section of floor.

Office Buildings.—This is an age of great and quick changes, and,

Mr. Lowinson. already, some of the older high office buildings are being converted into storage buildings, with loads far in excess of those for which they were designed; and, although the writer would be satisfied with a distributed loading of 65 lb. per sq. ft. if he were sure the building would never be used for any other purpose, he thinks a provision of 150 lb. per sq. ft. not at all too large.

Wind Pressure.—The writer believes that a clause should be inserted providing that, where the walls are other than curtain walls, and the skeleton does not proceed more than three stories in advance of the walls, temporary wind bracing (wire cables, etc.) should serve. In all possible locations, stiff knee-braces should be insisted upon at all column connections.

Foundations.—The writer would separate wet sand from soft clay. Quicksand, for instance, should never be used on which to found. Wet sand will frequently bear from 4 to 6 tons, as long as it is confined, and a restriction to 1 ton should not be made absolute. On the other hand, the writer would hesitate long before permitting a load of 6 tons on any but the hardest kind of gravel, and then only when it overlies rock. It is his practice to permit a load of 15 tons per sq. ft. on Portland cement mortar, and he allows only 10 tons per sq. ft. over Portland cement concrete.

Pressure on Wall-Plates.—For the pressure on wall-plates, the writer would use 200 lb. instead of 250 lb.

Shrinkage and Masonry.—Great trouble has been experienced with stone facings on brick walls because of the unequal shrinkage of the brickwork and the stonework; for this reason the writer suggests a class wherein stonework must be considered either as non-bearing, or it should pass entirely through the wall at certain distances in its height so as to get a proper bond. In case of shrinkage, in the former instance, the facing is held by galvanized-iron anchors.

Paragraph 37.—Timber Columns.—It is suggested that a more modern formula than the Gordon formula be used for timber columns.

Details of Floor Beams.—The writer would add that where a floor beam transmits a heavy load it should rest upon a girder if possible, instead of framing it on the web. Where the girder is composed of two or more rolled sections, and unless definite means are taken to transfer the load from one to another, the load should be applied on top if possible.

These specifications will serve very well as a standard for structural engineers. The writer would be pleased to see, included with such a set of standards, standards of practice in such matters as framing timbers, cutting stonework, differentiation of skeleton, cage and independent masonry wall construction. protection and preservation of materials of construction, other than those included by the

author, details of construction on piling, and protection against discoloration, efflorescence, etc. Mr. Lowinson.

EUGENE W. STERN, M. AM. SOC. C. E.—The speaker wishes to add, to those already expressed, his thanks to Mr. Schneider for this timely and valuable paper, and his appreciation of the labor involved in its preparation. Mr. Stern.

The past decade has seen a vast amount of building construction, in which the Engineer has taken a very prominent part; and wide and free discussion, by those having experience in this class of work, should prove of great benefit to the Profession.

As a whole, the speaker finds very little in Mr. Schneider's paper to criticise, and very much to commend.

The application of concentrated loads to beams, and the author's method of designing foundations for dead load only, is heartily approved; also, the schedule of requirements for steel, which, if generally adopted, would tend to simplify and make uniform the processes in the steel mills.

Mr. Schneider has emphatically condemned the use of cast-iron columns in building construction. In this the speaker must take issue with him. While he does not advocate their use in high buildings, or under any or all circumstances, he still believes that they may often be used profitably in buildings where wind bracing is not necessary; but only under thoroughly competent supervision and inspection.

The requirements of modern building construction, as well as machine building, have developed a higher state of the art than that which existed years ago when the bridge builder very properly condemned it.

The modern foundry has adopted better methods, and more is known about the proper mixing of iron, and making the moulds, as well as in designing castings. It is possible now, under proper supervision, to obtain satisfactory results, if the work is properly designed. The material should be of the proper mixture, and, in the design, proper consideration should be given to shrinkage. The castings should be properly stripped in cooling, and a sufficient number of test holes bored to determine whether or not the core has shifted.

It is the speaker's practice to insist upon the following conditions:

1.—A material which develops a strength of 2 400 lb. central load on a bar 1 by 1 by 12 in. with a minimum deflection of 0.1 in., and which is not so brittle as to break off when struck on an edge with a hammer;

2.—Column brackets or seats in which the metal is never more than ½ in. thicker than the column, preferably of equal thickness or less;

Mr. Stern.

3.—A maximum shear of 1 500 lb. per sq. in. for brackets, and 2 500 lb. in tension.

Mr. Schneider says that "beams are supported by lugs or brackets cast on the columns, thus producing eccentric loading and bending strains."

Eccentric loading, whether in steel or cast columns, must be considered, and the column section must be increased to take care of it. Very often, by properly balancing the loads, and by adjusting the position of beams, eccentricity can be largely reduced.

It would be an extremely difficult matter to legislate cast iron out of use. Would it not be better, therefore, to establish a high standard for its use, and thereby insure good results, than to ignore its existence as a material of construction?

The speaker hopes that Mr. Schneider will see fit to write a specification for cast iron. A specification for structural iron work for buildings is certainly not complete without it.

Paragraph 3, Live Load.—While it may be true that the actual weight of all persons who could possibly be assembled in a room with fixed seats would be not more than 40 lb. per sq. ft., still, there are occasions when these seats may be removed or decked over so that a greater load could be placed upon the floor. Vibration, also, should be considered. In view of this, the speaker does not think that 80 lb. per sq. ft. is any too much for public rooms with fixed seats. As regards the loads in assembly rooms, armories, ballrooms, etc., in which there are no fixed seats, it is the speaker's opinion, based upon observation, that a mass of people may weigh 80 lb. per sq. ft., and, considering that in all such places vibration, due to a mass of people moving, may occur, he believes that a load of 120 lb. per sq. ft. would not be too much.

Sidewalks in front of buildings should provide for a live load of at least 300 lb. per sq. ft.

Paragraph 12.—This paragraph appears to the speaker to be ambiguous. In the finished structure, are the walls and the partitions to take any of the wind load, or is the steel frame to take it all? If the latter meaning is intended, the speaker does not agree with Mr. Schneider. The finished walls and partitions of buildings may certainly be very efficient in withstanding wind pressure.

Paragraph 13, Foundation Loads.—The permissible pressure on foundations is, of course, a matter which requires careful consideration of the case in hand, and trained judgment.

In poor soil, a test should always be made. In some cases, where certain kinds of soft clay or fine wet sand are met, the pressure might have to be reduced to ½ ton per sq. ft.

Paragraph 14.—The speaker believes that the use of lime mortar should be prohibited in engineering work, as lime mortar is very

uncertain in hardening, and hydraulic cement is so cheap that it hardly pays to use lime. In fact, Portland cement is almost entirely specified by the speaker, to the exclusion of even natural cement, as being much more reliable and as costing very little more for the same strength of mortar. Mr. Stern.

Paragraph 16.—Regarding the carrying capacity of piles, the speaker believes that the surest and easiest way to determine what a pile will carry is to make an actual test on the ground, by driving test piles and loading them. There will then be no uncertainty.

Paragraph 45.—As regards tie-rods, the speaker believes that they should not be omitted, even if the concrete slab system of flooring is used, but that in this case they should be placed near the top flanges of beams.

If tie-rods are omitted, as some of the makers of patented concrete systems suggest, there is always the likelihood of an accident occurring during erection, before the floor system is put in. The speaker knows of two cases where beams, not being stayed with tie-rods, sprung sideways under a load, and caused serious accidents.

Paragraph 54.—Column splices should be designed to resist, not only bending strains, but twisting strains, in any direction, as this may come on them during erection.

There seems to be a tendency among some designers to cut the column splice down to the fine point, splice-plates are put only on two faces of the column, the angle lugs being omitted. The speaker does not consider this practice good.

The author says nothing regarding erection. The speaker believes that the supervision of the erection of the work which he has designed is one of the most important of the Engineer's duties, and thinks that the author might find a place for it in his specifications, to make them complete.

Many bad accidents have happened because proper supervision of the erection has not been given to work which has been properly designed.

The Engineer should have absolute control and supervision over the work which he has designed, and should have, besides, the power to pass on some things which are really outside of his own work, but which may affect its safety.

The erection of the steel, as a rule, is coincident with carrying up the walls and putting in the floors. Other trades will be at work on the building, and great care and vigilance must be exercised to prevent overloading the floors and straining the structure.

The speaker has seen a pile of cement, to be used later in floor arches, weighing 400 lb. per sq. ft., piled on a section of floor designed to carry a gross load of 125 lb. per sq. ft. Also, on the same floor, a pile of steel bars, to be used in re-inforcing the concrete floor, weighing 300 lb. per sq. ft.

Mr. Stern. Stone-setters, if not prevented, frequently attach booms to the columns of a building to hoist their stone to position. Hoisting engines are often placed on the first floor, and cause a great deal of vibration. They should either be placed in the cellar or shored up so that they clear the ironwork entirely.

Boom derricks, for erecting the ironwork, will probably require special temporary vertical bracing between the columns in those panels immediately adjoining the derrick. The stress which these derricks produce on the uncompleted structure is often much greater than that due to wind pressure.

Mr. Darrach. CHARLES G. DARRACH, M. AM. SOC. C. E. (by letter).—If Mr. Schneider had presented his paper eighteen or twenty years ago much error would have been avoided, and money and lives would have been saved. The paper is valuable in many ways, and calls attention to the practice at the present time.

To obtain the best results, capitalists must be made to recognize that the structural and mechanical engineers, as well as the architects, are absolutely necessary, and that it cannot be expected of the architect that he shall be an encyclopedia of knowledge.

The capitalist employs the architect at a flat 5% or less, and the architect (generally), in order to obtain a fair remuneration—or sometimes, even, make ends meet—depends for the most important details, which he could only get by association with engineers of knowledge and experience, either upon competing contractors, or upon the immature efforts of some ambitious graduate of a so-called scientific school.

The paper is admirable, yet it may be suggested that any hard and fast rule for the safe load on piles should be deprecated; the experience gained by the failure of the pile foundations under the western approaches of both the South Street and the Chestnut Street Bridges, over the Schuylkill River, in Philadelphia, Pa., are sufficient examples, and no pile foundations should ever be attempted without full knowledge obtained by ample and careful borings.

Although the paper, to a certain extent, refrains from discussing structural concrete, engineers should unqualifiedly condemn the common practice of using so-called cinder concrete. This cinder concrete is neither strong, water-proof, nor fire-proof, and hard-coal ashes are often substituted for cinders.

Floors should be made either of hard-burned brick, laid in Portland cement, or of good honest concrete made of trap or some other stone not affected by heat.

In a well-designed office building, the floor structure should be so designed that partitions can be erected in any location—or changed at the pleasure of the management. This condition places the partitions among the items making up the live load.

E. C. SHANKLAND, M. AM. SOC. C. E. (by letter).—Mr. Schneider is to be complimented for presenting a paper so valuable, so exhaustive and so carefully worked out. He will have the thanks of all the members of the Society who have to do with structural work. Mr. Shankland.

In this paper, it is difficult to find anything to criticise, or in which one differs from the ideas set forth and the conclusions drawn.

As regards live load, the amount provided for in the specification is ample. In 1891 the well-known contractor, the late George A. Fuller, weighed all the furniture in the main room of his offices in the Rookery, and also took the weights of a number of people whom he invited in, more than for whom he could provide seats, and more than had ever before been in the office at one time. The total weight was less than 10 lb. per sq. ft.

Mr. Fuller was very much opposed to the live load, as ordinarily considered. He favored a concentrated load on joists and girders, but did not favor carrying this load into the foundations. The writer does not believe it possible to get a uniform settlement with foundations not on solid rock, if live load is considered in designing them.

In the writer's opinion, the wind pressure called for, 30 lb. on the steel frame, is too high. The greatest velocity observed, at the office of the United States Weather Bureau in Chicago, since its establishment, was on February 12th, 1904, when a velocity of 115 miles was recorded, but the velocity for a 5-min. period was only 84 miles.

Some years ago the U. S. Weather Bureau re-rated their anemometers, and, disregarding the old Smeaton rule for determining the pressures, worked out a new formula, which they now use. According to this, the true velocity for an indicated velocity of 80 miles is 62.2 miles, and the corresponding pressure is 15.5 lb. per sq. ft. Other velocities and pressures are in the same ratio. Certainly, if the steel frame is designed to resist the ordinary wind pressure, the masonry walls, floors and partitions may be relied upon to enable the structure to withstand any greater wind which may occur.

Paragraph 7 should include the weight per square foot of skylights. In the roofs of courts there is generally nothing but glass and the steel frame.

The working pressures on masonry are too low. Hard-burned brick in Portland cement mortar will stand 20 tons safely; and Portland cement concrete properly mixed is good for 30 tons per sq. ft.

The author rules cast iron out of the specification on account of its very poor resistance to bending and tension. Cast-iron column bases are used quite generally, and are of much greater diameter than the columns they support, but the author must have had in

Mr. Shankland. mind only columns. Cast-iron columns should never be used where they have to resist either bending or tension, but there are many warehouses, several hundred feet square, and from 6 to 10 or 12 stories high, where the columns have to carry only a compression load, and in such cases the use of cast iron seems to be perfectly permissible. In many cases cast-iron columns can be had in very much less time than steel columns, and this renders their use desirable, even if difference in price is not considered.

It is not always possible to keep the depth of beams within one-twentieth of the span. Paragraph 45, limiting the deflection of beams to $\frac{1}{30}$ in. per foot of span covers the case and renders Paragraph 33 unnecessary.

For reasons before stated, in Paragraph 35 should be inserted the specification of the American Bridge Company for 1901, Paragraph 114.

In Paragraph 40 the writer thinks adjustable members for wind bracing and counters are preferable in building work; although they should be avoided in bridge work.

In a bridge, when the false work is taken out, the bridge settles down and takes up the play in the members, but in the case of a building no such condition obtains.

The same reasoning, if correct, applies to Paragraph 83.

Mr. Crowell. FOSTER CROWELL, M. AM. SOC. C. E. (by letter).—The introduction by the author of the feature of concentrated live loads on floors opens the way, as has been shown, for a logical and important reduction of assumed column loads, in the case of office buildings and apartment houses, the rational extent of which should be determined within consistent limits. The rule adopted for the reduction in the proposed specifications, Paragraph 10, is the current and official interpretation of the New York Building Code, Sec. 130, as therein stated. A comparative analysis of its application with that of a somewhat different interpretation is made in the accompanying table, which is simple and self-explanatory. The second interpretation is based, not on the theory that the actual floor loads would diminish from floor to floor, as the language of the rule would imply, but in consideration of the fact that the chances of full loads occurring simultaneously on every floor grow less and less as the number of floors increases; for present purposes, however, the arbitrary limitation of reduction to 50%, as in the rule, is retained.

By reference to the table it will be at once noted that while a considerably greater reduction results from the second interpretation of the rule, the extreme effect is still well above (on the safe side of) the limit accepted by the author, on page 381, and its "average total" agrees still more closely than the other with the investigations of Blackall and Everett, to which he refers.

A little simple figuring from the table shows that the resulting saving of column section would amount to 14% in the case of ten stories, 20% in twenty stories and 21% in thirty stories. Mr. Crowell.

Rule "A." Successive Live Floor Loads Reduced, Below the Top Floor, 5% Each, until the Reduction Factor is 50 Per Cent.	Rule "B." Successive Live Column Loads Reduced, Below the Columns Supporting the Top Floor, 5% from the Nominal Total Live Loads, Until the Reduction is 50 Per Cent.

Situation.	Floor Loads, in Pounds per Square Foot.					
	Successive "A."	Total "A."	Average "A."	Percentage.	Total "B."	Average "B."
Roof	50	....	....	....	50	....
1 (top floor)	40	90	40	100	90	40
2	38	128	39	95	126	38
3	36	164	38	90	158	36
4	34	198	37	85	186	34
5	32	230	36	80	210	32
6	30	260	35	75	230	30
7	28	288	34	70	246	28
8	26	314	33	65	258	26
9	24	338	32	60	266	24
10	22	360	31	55	270	22
11	20	380	30	50	270	20
12	20	400	29.16	....	290	20
13	20	420	28.46	....	310	20
14	20	440	27.86	....	330	20
15	20	460	27.33	....	350	20
16	20	480	26.87	....	370	20
17	20	500	26.47	....	390	20
18	20	520	26.11	....	410	20
19	20	540	25.79	....	430	20
20	20	560	25.50	....	450	20
21	20	580	25.24	....	470	20
22	20	600	25.00	....	490	20
23	20	620	24.78	....	510	20
24	20	640	24.58	....	530	20
25	20	660	24.40	....	550	20
26	20	680	24.23	....	570	20
27	20	700	24.07	....	590	20
28	20	720	23.93	....	610	20
29	20	740	23.79	....	630	20
30	20	760	23.66	....	650	20

Note.—The Roof Load is not taken into account in computing the averages of the floor loads in this table, but is included in the totals.

An inquiry into the inherent practicability of imposing even such reduced loads throughout a modern office building is of interest. If, for example, half the average live load is assumed to be due to furniture, safes, etc., and the remainder, or 10 lb. per sq. ft. of floor, to people, we can compute the number of persons of given average weight whom it would be necessary to introduce into and distribute through the building to make up the assumed total load on all the floors. The writer has made some observations of what may be termed the passenger capacity of some typical office build-

Mr. Crowell. ings. Case I is a prominent and exceedingly populous building on lower Broadway in New York City, 16 stories in height above the street and occupying about 200 by 150 ft. of ground area. Assuming that 80% of the area is available floor space, the aggregate for the 15 floors dependent on elevator service is 360 000 sq. ft., and this, multiplied by 10 and divided by 150, representing the average weight per person, gives 24 000 as the number of persons who would have to be crowded into this particular building at one and the same time in order to subject it to the assumed loading. There are eight elevators, each with a crowded capacity of 12 persons besides the operator, capable of making 25 round trips each, or 200 in all, per hour. The total hourly passenger capacity thus would be 2 400 in one direction, and, to distribute 24 000 people would require 10 hours, during which period none would descend, which is a forced assumption and absurd on the face of it, but is essential to the completion of the loading. Case II (hypothetical) is a 10-story office building 100 by 100 ft., with 72 000 ft. of available floor space and 4 elevators each holding 12 passengers and making 30 trips per hour, making the combined hourly capacity 1 440 persons in one direction. In this case, on the same basis as before, 12 lb. per sq. ft. would be due to people, the number of persons to be distributed would be 5 760, and the time required to distribute them through the building would be 4 hours, with no persons descending.

While this comes nearer to being a supposable case than the other, it is evidently so improbable as to be practically impossible, and the conclusion seems to be warranted not only that the first interpretation of the rule is too conservative, but that for office buildings more than ten stories in height the limit of the second interpretation might consistently be lowered materially.

The columns which support stairways, main corridors, elevator landings and other parts of the building which are subject to heavy concentrations of people in case of the occurrence of a panic, from fire or otherwise, should not have any reduction, but, on the contrary, should be specially designed to meet properly the maximum concentrations, which might occur simultaneously on all the floors. Nor should there be any reduction in the case of warehouses, department stores or assembly halls.

The author is clearly correct in the reason given for the failure of some foundations proportioned for theoretical live loads which never occurred, and his suggestion to proportion for dead loads only, using a lowered modulus, is scientific, practical and a distinct advance. There is another category of unequal settlements to be guarded against, however, where the foundation loads may be practically identical, but where there are abrupt variations in the bearing power of the soil underlying adjacent parts of the same build-

ing. Such differences frequently manifest themselves, during the progress of the work or previously, and can be readily compensated, but, in soils requiring pile foundations, especially, suitable adjustment of the foundation must be provided for in advance, based upon a careful determination of the variations by means of test-piles or otherwise. In cases where great variations of this nature occur, the pile-driving formula adopted by the author, Paragraph 16, must be modified to meet the conditions, or unequal settlement, in greater or less degree, will ensue. Mr. Crowell.

The reason for this has already been pointed out by the writer, in a paper presented before this Society,* but may be briefly repeated here. The effect of the introduction of the constant increment in the denominator is inevitably to disproportion the rate of loading, causing it to be relatively less in the firmer soil and relatively greater in the more yielding, whereas the reverse should be aimed at.

As before stated, the writer regards the formula in question as safe and suitable for equal conditions. The suggestion is here offered that it would be well to expand Paragraph 16 to take in a steam-hammer formula, and also to cover concrete piles, now coming into extensive use, and to which the limitations of wooden piles do not apply. Usually, the concrete piles are not "driven" in the sense that wooden piles are, and it would seem important, taking this fact and others which suggest themselves into consideration, to formulate correct rules governing their use.

In the opinion of the writer, Paragraph 54 should be extended; he fully agrees that in dry situations the protection given to steel by the concrete is sufficient, and that, in such cases, the steel need not, and should not, be painted; but where subjected to alternate wet and dry conditions, or where the steel may not be completely surrounded and insulated by concrete in adhesion, further and more active measures should be taken than simply to omit the painting. In his own practice, the writer has specified as follows:

"Such (steel) plates, and other steel members which are to be entirely imbedded in the concrete, must be carefully washed with acidulated water of sufficient strength to remove all grease and rust, and be well scrubbed; they shall then be washed with a hose jet of clean water neutralized with lime-water, and painted with a thin coat of neat cement just before the final covering with the concrete, which must be mixed and disposed so as to insure complete contact throughout with the steel. Steel members only partly imbedded in concrete, or in situations exposed to moisture acting through the concrete, must be treated, before leaving the shop, with two coats of * * * after having been thoroughly cleaned, as specified above."

---

* "Uniform Practice in Pile Driving." *Transactions*, Am. Soc. C. E., Vol. XXVII, pp. 106, 596.

Mr. Clarke. ST. JOHN CLARKE, M. AM. SOC. C. E. (by letter).—It is gratifying, to those who have spent some time and effort in what has been called "Architectural Engineering," that an engineer of Mr. Schneider's attainments and prominence should present a paper on this subject so elaborately and carefully prepared. This one thing will do as much to elevate and dignify this branch of work as the subject-matter of the paper itself. Unfortunately, building work has been left largely to the "building trade," and has been neglected by the general engineering profession, to the disadvantage alike of buildings and of the Profession.

Comparing the author's specifications with the New York Building Law, it is evident that, for office buildings with the usual column spacing, the floor beams would be much heavier, the girders lighter and columns about 20% lighter, with greater uniformity in sizes and connections. A concentrated load of 5 000 lb. seems to be somewhat excessive. A safe weighing 5 000 lb. will rest on four points of support, and cannot be considered as a concentrated load on one point. With floor beams 20 ft. long, and spaced at 5-ft. centers, a concentrated load of 4 000 lb. is equivalent to 80 lb. per sq. ft., which, it would seem, is an ample allowance. The New York law requires 75 lb. per sq. ft. for live load, and this has given very good results in practice. The assumption of a concentrated load is more rational than the commonly assumed uniform load per square foot of floor, but the writer would make it less than 5 000 lb.

The author's requirement that the depth of beams and girders should be limited to one-twentieth of their span does not seem better than the old rule that the deflection shall be limited to $\frac{1}{30}$ in. for each foot of span. It is often impracticable to use beams or girders of only one-twentieth of their span.

The use of cast iron should certainly be discouraged; but, with discretion, it may be used for lintels, column bases, and even for columns in some instances. Cast iron, therefore, should receive some mention in tables giving permissible strains. The design of cast-iron bases is often most execrable.

In the matter of wind pressure the author is not quite clear. The writer interprets the specification as meaning that the steel frame is to be considered as an independent structure, subject to a wind pressure of 30 lb. per sq. ft. on its own area, and that the building, as a whole, is to be considered as subject to the same wind pressure, with walls, partitions and floors contributing resistance. This is a very broad requirement, and it would be hard to find two men to agree on its application to any one building. The conditions are peculiar to each case, requiring the exercise of care and judgment. The writer has frequently used 20 lb. per sq. ft. on the exposed surface of the building, considering the steel frame to sup-

ERRATUM.

Transactions, Vol. LIV.

Page 471. After the 23d line insert the following line:
"author has done the Engineering Profession a practical service by"

Mr. Crehore's first sentence then will read: "The author has done the Engineering Profession a practical service by laying before it in a clear and concise manner a subject upon which there are wide differences of opinion, and by suggesting a specification which is more uniform and is better adapted exclusively to buildings than any other heretofore proposed."

ply all resistance, and increasing the permissible unit stresses 50% for the combined strains due to wind and other loading. This was done in deference to the requirements of law or precedent. In his judgment, this wind pressure might be reduced somewhat farther. If this were not true, many buildings, which are at present standing, would have fallen some time ago. Mr. Clarke.

The use of the straight-line formula is to be commended for columns. Why should it not be used for wooden columns, also?

In Paragraphs 41 and 56, referring to symmetrical sections, attention should be called to the importance of balancing properly the connecting rivets about the center line or neutral axis.

The rules for spacing tie-rods and separators are rather arbitrary.

The allowable compressive values for concrete seem to be entirely too conservative. These values are an inheritance from the time when cement was not as good or as uniform as it is at present. From recent tests, it would seem that Portland cement concrete could be safely loaded to 35 tons per sq. ft. It may be objected that concrete of first-class quality cannot be depended upon. This objection is well taken, if the work is not done under proper supervision, but the use of these specifications would seem to require the services of an engineer, and expert supervision should be expected for all work of this character.

WILLIAM W. CREHORE, M. AM. SOC. C. E. (by letter).—The laying before it in a clear and concise manner a subject upon which there are wide differences of opinion, and by suggesting a specification which is more uniform and is better adapted exclusively to buildings than any other heretofore proposed. The subject is a very important one, and it is to be hoped that engineers will become sufficiently interested to work for the adoption of a uniform specification, which, if not indeed above criticism, will at least be entirely within the limits of good practice. Mr. Crehore.

Probably there is a greater divergence of opinion among authorities on the subject of assumed live loading for floors than upon any other subject. As rain descendeth alike upon the just and upon the unjust, so municipal ordinances are made to govern both the skilled and the unskilled, and no discretion can be permitted to those whose knowledge of matters pertaining to building construction is perhaps of a higher order than that of the authors of the law. These laws usually err on the safe side, and are purposely made so. The writer's observation confirms that of the author and the authorities quoted, that the weight of an ordinarily crowded assemblage of people seldom if ever exceeds 40 to 50 lb. per sq. ft. This load, of course, is likely to occur not only in theatres, churches, schoolhouses and such buildings, but also in office buildings and hotels,

Mr. Crehore. and even in dwelling-houses. That is, whether a building is for public assemblages or not, it is likely to be loaded on rare occasions with throngs of people over small portions of its floor area; but, simply to provide for the dead weight of the live load (if the expression may be allowed) is admittedly not enough. A live load implies movement of some kind, and should be provided for according to its kind. The author covers this point by recommending that this load be doubled for places where strong vibrations may be expected, such as ballrooms, drillrooms, gymnasia, etc., and by limiting the span of the beams and girders to fifteen times their depth. The writer would make some increase in all classes of buildings, on the ground that dwelling-house and apartment-house floors, and the like, are occasionally subjected to vibration under their full load. The author's provision for a minimum concentrated load, of course, takes care of this point for all the shorter spans.

It is very easy to be deceived by one's observation in estimating the density of a crowd of people, because of the fact that their distributed weight per square foot varies inversely as the square of their distance apart. Assuming the average person to weigh 150 lb., if a crowd of people stood so close that each one occupied just 1 sq. ft. of floor space, the live load would then be 150 lb. per sq. ft. It is possible to squeeze people as close or even closer together than that, and there are congested localities in New York where this load frequently occurs. Now, assuming this assemblage to be spread out so as to become a 50-lb. live load, then the little square occupied by each person will contain 3 sq. ft. instead of 1, and will measure 1.732 ft. or 1 ft. 9 in. on a side. In other words, each person is now only 9 in. further from each of his neighbors in every direction, and yet the distributed load is reduced to one-third of its former amount. The following table gives the weights per square foot for crowds of different densities, varying by inches from 12 to 24 in., each person weighing 150 lb.

12 in. apart	150 lb.	19 in. apart	59 lb.
13 " "	127 "	20 " "	54 "
14 " "	110 "	21 " "	49 "
15 " "	96 "	22 " "	44 "
16 " "	84 "	23 " "	40 "
17 " "	75 "	24 " "	37½ "
18 " "	66 "		

The author's treatment of this whole subject leaves little to be desired. There is this about it, however. The very fact that it makes the computations for a building a little more complicated than the method at present in vogue, so that every architect's office boy will not be able to select the floor beams from the mill hand-

books with reasonable exactness, will be likely to mitigate against its adoption by most municipal authorities. The tendency of municipal governments in this direction is to simplify and tabulate requirements for the use of the general public, and, where complications and ambiguity are likely to creep in, to select the alternative offering the greatest safety, regardless of economy. Until such time, therefore, as all work is required to be executed under the design and supervision of a licensed engineer or architect whose license depends upon his competency, the public welfare requires the existence of rigid, non-elastic legal regulations which can be easily applied by the layman, without the possibility of jeopardizing the safety of construction by errors of omission or a misapprehension of the requirements. This fact, however, need not and should not prevent the adoption by engineers of a set of rational requirements aimed to secure a thoroughly uniform practice. A substantial agreement between engineers must certainly be brought about first, before any uniformity may be expected among the different municipal law makers. **Mr. Crehore.**

In regard to the effect of vibrating loads on the structural frame of a building, something should be said about the very important part played by the floor construction itself. The writer agrees with the author that the steel frame of a building should be self-contained, that is, not dependent upon the other parts of the construction for rigidity or assistance of any kind. Nevertheless, a great deal of aid is given to the steel structure in the performance of its work by the walls and the fire-proof floor construction; and, in considering the subject of live loads, the ratio of the live to the dead load plays an important part. A vibratory load will impart motion to the supporting floor only after the inertia of the floor's dead weight has been overcome. In other words, only that portion of the impact which is not absorbed by the dead weight of the floor construction is left to impart motion to the structure. Thus, it will be seen, that a 40-lb. live load vibrating on a 35-lb. floor will affect the floor system, and consequently the beams and girders, far more than the same load would affect a 70-lb. floor. Yet even the 35-lb. floor will absorb some of the vibration, and it does not seem rational to compute the floor beams for the full effect of the vibration regardless of the inertia of the floor construction. The writer advocates adding to the live load a percentage for impact which shall vary with the ratio of the live to the dead load, according to some law to be determined either rationally or empirically.

As long as the load of a throng of people is assumed at 70 or 80 lb. per sq. ft. such refinement as the above is of no consequence at all; but, when it is proposed to assume the live load very close to what it actually is, it then becomes exceedingly important to take

Mr. Crehore. account of the effect of impact. Moreover, the effect of impact is very largely controlled by the dead weight of the floor construction itself. That these facts have been known all along, but perhaps not thoroughly realized, probably accounts for the wide divergence of opinion among municipal authorities on the subject of live loads; this divergence being primarily due to the different methods used to provide for this phase of the subject. The author's method of providing for it is eminently safe, and at the same time economical, but, in the writer's opinion, it is not fair to the fire-proofing companies, whose material forms the bulk of a floor's dead weight, to discriminate in favor of a light-weight floor construction, which is now a custom that Mr. Schneider's specification does not propose to amend. Not only the weight of the construction, but also the manner of laying it and the degree of completeness with which it is bonded with the beams and girders to form a homogeneous whole, are of great importance—in buildings as in bridges—in considering the effect a vibrating load has on the beams and girders; and a floor system which possesses these helpful qualities should not be discriminated against on account of its weight only, as is now done. By this the writer does not intend to convey the idea that there are no other considerations which should prevail in the choice of a floor construction, but merely to direct attention to the fact that the heavier floor is usually of more service than the lighter one in taking care of a vibrating load. The heavier floor, therefore, should be credited with a greater ability to absorb impact than the lighter floor has. This being done, a result might be obtained something like this:

	Heavy floor.	Light floor.
Actual dead weight of floor per square foot.	70 lb.	35 lb.
Assumed live load per square foot.........	40 "	40 "
Allowance for impact, per square foot.....	5 "	40 "
Total	115 lb.	115 lb.

This illustration is not given with any confidence that the allowance made for impact is proportionally correct, but merely to show that the net result affecting the floor beams might be identical in two cases where the dead loads were very far apart. Under this rational treatment of the calculations, therefore, much of the advantage apparently possessed by a comparatively light floor system must be surrendered. The same floor beams would be used in one case as in the other.

If, then, a tall building is properly constructed, a vibrating load should affect the beams very little, the girders even less, and the columns not at all. The excess added for impact (by whatsoever

method), therefore, may be entirely disregarded for the columns and beyond. In addition to this, the author makes the very reasonable reduction of live load on columns amounting to 5% for each story until a reduction of 50% is reached. The writer agrees with this, but would go further, as just stated, and first deduct the whole original allowance for vibration or impact, before beginning to figure the 5% deductions. Mr. Crehore.

Coming now to the foundations, it becomes evident that they will be affected by the live load least of all. The live load being a shifting quantity, constantly being transferred, and the total present at any one time actually being comparatively small, its share of the combined effect upon a yielding soil is seen to be unimportant. The author's method of proportioning the foundation areas for dead loads only, therefore, is about the only logically correct way to do it, under a legal requirement which includes the live load in the bearing unit. The writer has practiced this method for ten years or more, and is on record, as recommending it for general use, as long ago as 1896.*

It would be well to limit the use of cast-iron columns with side-bracket connections to the minimum size of an 8-in. column by 1-in. metal, permitting no smaller column unless its whole load is to be placed squarely on the top of the column. The maximum permissible load on side-bracket connections should also be specified, depending upon the thickness of the column, since most failures of cast brackets show the brackets to have pulled metal out of the main shaft, rather than to have broken off. The chief objection to cast iron is the difficulty in connecting to it in a way that will insure rigidity and stiffness. To connect an adequate system of wind bracing to cast-iron columns properly is an impossibility; and, because of the lack of rigid connections, the cast-iron-column building stands in greater need of wind bracing than the steel-column building. It is a worthy aim to make the structural frame entirely self-contained, yet it may not always be the economical way, for there is no denying the fact that well-built 12-in. and 16-in. curtain walls are great stiffeners; and, as they are to be in place anyway, they might as well be made of some use. The writer has sometimes been obliged to compromise in this matter, and has built several eight-story warehouse buildings of cast columns half way up and steel in the upper four stories, with excellent results. This permits putting in the stiffening where stiffening is most needed; and the heavier walls and columns of the lower part of the building lower its center of gravity, the walls providing ample filling to prevent vibration in the lower columns. This scheme also avoids the use of the smaller sizes of cast-iron columns.

*In a chapter contributed to Professor Du Bois' book, "Stresses in Framed Structures."

Mr. Crehore. As to the bearing power of piles, a maximum load of 20 tons will place the limiting height of a building on pile foundations at about twenty-eight to thirty stories. Piles cannot be properly driven and cut off if spaced much closer than 30 in. from center to center. If spaced 27 in. apart they will sustain the same load per square foot of ground area, allowing 20 tons per pile, as is permitted by the New York law for the unit bearing on good earth, namely, 4 tons. For their respective kinds of foundations, the Park Row Building and the St. Paul Building stand at about the limiting height possible under the present New York Code. To specify the maximum load at 20 tons for any kind or size of pile, seems to the writer to be rather unsatisfactory. It puts a premium on small piles. To be sure, the formula takes cognizance of the size of a pile, in a way, because, under the same blow, the larger pile will take less set than the smaller one. But the formula is based on the friction bearing of the pile, and, in the many cases where piles penetrate to hardpan or rock, the formula is inapplicable. There, the 20-ton limit applies. Certainly, a large and a small pile are not of equal value under these circumstances, nor is it a sufficient provision to limit the pile's bearing power by its average cross-section, regardless of its length or the kind of timber of which it is composed. In the writer's experience, he has had to deal with more cases where the formula was not applicable than where it was. It seems, therefore, that, in driving piles, a large amount of both common sense and good judgment are requisite, and any rules or formulas are of very little practical value, except under certain particular conditions.

Mr. Turner. C. A. P. Turner, M. Am. Soc. C. E. (by letter).—The writer has read Mr. Schneider's paper with much interest, and only regrets that the author did not offer a specification for the truly modern type of building, namely, concrete-steel.

At present, the writer is constructing buildings of this type in competition with wood mill construction; and a comparison of the actual test loads which he guarantees to place on the finished floors for the various types of buildings, without damage to the construction, or evidence of undue weakness, may be of interest:

> Ordinary apartment-house floors, costing less than the old-style tile, with the steel frame left out, from 400 to 600 lb. per sq. ft., as may satisfy the owner;
>
> In warehouses, for which, in the older forms, the owner would be satisfied with a figured safe load of 500 lb. per sq. ft., the writer agrees to place a test load of 2 000 lb. per sq. ft., without damage to the construction, or evidence of weakness; and, in the last building on which he bid, the figure was the same as for wood construction calculated for 500 lb. per sq. ft., and a factor of safety of 4. The floor area of this building was 9 acres in nine floors, or 1 acre per floor.

Naturally, in dealing with a type of construction in which the strength can be doubled with an increase in cost of 10%, or less, anything of the nature of a light or skinny specification for floor loads seems to be out of place, and it is natural to inquire whether the owner will continue long to prefer to pay 50% more for a type of construction which is only 20% as strong as the newer form, for no better reason than that heretofore it has been the custom. Mr. Turner.

C. C. SCHNEIDER, PRESIDENT, AM. SOC. C. E. (by letter).—In reviewing the discussion, the writer wishes to emphasize what he said in the preface, that his specifications were intended for the use of the educated engineer. The designing of the structural work of buildings should be entrusted to experienced specialists only, and the specifications were prepared with that end in view. The writer, therefore, has carefully refrained from introducing any tables, rules or formulas, which, if put into the hands of inexperienced men, would be dangerous expedients, productive of mischief, as has already been demonstrated in many cases. Neither has he gone too much into the details of design, as this should be left to the judgment of the designer. Mr. Schneider.

Building laws have failed to protect the public from disasters caused by poor designs made by inexperienced men. They generally go too much into details, thereby assisting incompetent men to design the structural work of buildings without a knowledge of mechanics, statics or even the first principles of designing. We would certainly obtain better structures if the responsibility rested entirely on the designer, the same as in bridge construction. This is probably one of the reasons why the designing of structural work has not kept pace with that of railroad bridges, and generally does not receive the same rational and careful attention.

Slipshod methods are yet the order of the day. Many designs are made in a perfunctory manner, to conform in a general way to the building laws, but critical points very often receive little or no attention. Thus we see buildings, in the construction of which enough material is wasted to make them first class and entirely safe for all possible contingencies, with some details so weak that it is surprising they stand up under their own weight, unless it is by force of habit.

Paragraph 1.—Dead Loads.—Messrs. Post, Darrach and Freitag suggest that provision be made for the weight of movable partitions in office buildings.

The writer believes that 10 lb. per sq. ft. would be sufficient to provide for this, which should be added to the dead load of the floor.

Mr. Cooper suggests that allowance be made for the dead weight of the floor, as heavy solid floors should have an advantage over those of light weight.

Mr. Schneider. Mr. Crehore also calls attention to the fact that heavier floors should be credited with a greater ability to absorb impact than lighter ones, and suggests that an impact be added varying with the dead weight of the floor.

The writer, acting on the above suggestions, has, in his revised specifications, limited the dead load to be assumed for fire-proof floors to 75 lb. per sq. ft. for the floor system, which is practically the method proposed by Mr. Crehoré.

Mr. Post suggests including a table of the approximate weights of all the well-known systems of floor construction. If he had submitted such a table in his discussion it would have been a very valuable contribution. The writer has the approximate weights of different kinds of floor construction, but cannot be induced to publish the same, wishing to avoid controversies, as he has not been able, by his own calculations, to reduce the weights to those given by some of the manufacturers.

Paragraph 3.—Live Loads.—In reference to the live loads which should be specified for the various classes of buildings, the opinions of those who have discussed this paper do not seem to be unanimous. Messrs. Cooper, Freitag, Darrach, Clarke, Shankland, Crehore, O'Brien and Aus think the specified loads are ample.

Messrs. Clarke and Aus think the concentrated load specified for office buildings excessive.

Mr. Worcester suggests a distributed load of 50 lb. per sq. ft. for dwellings, hotels, office and all other public buildings.

Mr. Smith would use a distributed load of 75 or 80 lb. per sq. ft. for all assembly rooms, office buildings, ordinary stores and light manufacturing buildings, in connection with the concentrated loads specified.

Mr. Post recommends for schools, theater galleries and churches, and for office buildings above the ground floor, 60 lb. per sq. ft; for the ground floors of offices, 80 lb.; for assembly rooms, armories, etc., 100 lb.; and for sidewalks, 250 to 300 lb. per sq. ft.

Mr. Blakeley thinks that no buildings should be figured for less than 80 lb. per sq. ft.

Mr. Clermont thinks 150 lb. per sq. ft. should be specified for all public buildings.

Mr. Lowinson recommends a minimum live load of 65 lb. per sq. ft. and 125 lb. for assembly rooms, 250 lb. for carriage-houses, 350 lb. for sidewalks, and 150 lb. for office buildings.

Mr. Stern suggests 80 lb. per sq. ft. for assembly rooms with fixed seats, and 120 lb. per sq. ft. for armories, ballrooms, etc., and 300 lb. per sq. ft. for sidewalks.

Mr. Johnson suggests for assembly rooms, armories, ballrooms, etc., a uniform distributed load of 150 lb. per sq. ft., with allowance for impact.

Mr. Schneider.

Mr. Smith and Mr. Lowinson think there should be no distinction between live loads for assembly rooms with fixed seats and those without.

The writer is surprised to find such a great variety of opinion on this subject, a variety corresponding to the vagaries of the provisions of the various building laws. As some of those who have criticised the live loads do not seem to comprehend the principles which guided the writer in adopting them, some additional explanations may be necessary.

Small differences in the assumed live loads are of far less importance than the rational design of details and the individuality of the designer.

Some of the discussors have discovered that it is possible to obtain greater loads than those specified. The writer wishes to state that he was fully aware of all those possibilities of loading, and, also, had carefully studied all the literature on the subject of experiments on the weights of crowds of people, mentioned by Mr. Johnson, before he ventured to write his paper, and has given all these facts due consideration in determining the live loads to be specified for buildings. After reading the discussions, he has again gone over the whole ground, has carefully considered all suggestions, and has modified the table of live loads in accordance with those criticisms which appealed to him as being well taken. The writer, in determining upon the live loads of buildings of various kinds, as stated before, has been guided by what is now considered the best practice in bridge building. It is not a question of how much load of any kind it is possible to pile on a square foot of floor area. The question is to make the buildings absolutely safe without wasting large quantities of material in places where it is not needed. A structure should be proportioned for a working load with the ordinary unit strains, and provision made for a congested load with unit strains well within the elastic limit, so that the structure may yet be safe under such extraordinary conditions of loading. The working load should be the probable maximum load which may be reasonably expected to occur, while the congested load is a load which is improbable, but within the reach of possibility.

The footwalks of the heaviest city highway bridges are designed for a live load of 100 lb. per sq. ft., yet there is a greater possibility of getting a congested load of 150 lb. per sq. ft. on the limited space of a footwalk than there is in filling a public hall with a crowd of people of the same density.

Railroad bridges are now generally designed for a possible future increase in the weight of locomotives and rolling stock, but no engineer would think of designing a long-span, double-track bridge under the assumption that both tracks will be entirely covered with

Mr. Schneider. the heaviest type of locomotives, or cars carrying heavy ordnance, and using the ordinary unit strain for that condition.

The writer desires to emphasize the fact that such unusual and extraordinary conditions of loading have been considered in his specifications, in assuming that it is rational and unquestionably good practice to allow a unit strain of 25% in excess of the ordinary working strain, or 20 000 lb. per sq. in. for congested loads, and 50% in excess, or 24 000 lb. per sq. in., for very extreme cases.

There are railroad bridges in existence now, some members of which are strained to 24 000 lb. per sq. in., including impact, almost every time a train passes.

Mr. Johnson has demonstrated that it is possible to squeeze in a limited space of 6 by 6 ft. enough able-bodied men to produce a load of 180 lb. per sq. ft. To make these experiments of greater practical value, it is suggested that they be extended to larger areas such as an ordinary-sized assembly room, say, 40 by 60 ft. Such excessive loads can only be produced by a panic, and then only on a limited area, and it is just such extreme cases of excessive loading on limited areas that the specifications are intended to cover.

Thus we find the specified concentrated load of 5 000 lb., with the usual beam spacing of 5 ft., to be equivalent to the following uniform loads per square foot of floor area: 400 lb. for beams of 5 ft. span, 200 lb. for 10 ft. span, 133 lb. for 15 ft. span, and 100 lb. for 20 ft. span. For spans of more than 20 ft. the uniform load of 100 lb. per sq. ft. will govern.

In *Engineering News* of May 25th, 1905, a correspondent states his observations on the Duluth Ferry Bridge:

"The equivalent net floor space, * * * is nearly 1 400 sq. ft., and a crowd of 814 people, including ladies and small boys * * *, would indicate a load probably of not more than 75 lb. per sq. ft.; yet the crowd was so dense that the superintendent—a slender, active man, whose authority was known—spent 1¼ minutes forcing his way the length of the car, or 50 ft., through the crowd, to open the gates."

Therefore, 80 lb. per sq. ft. may be considered as the maximum weight of a promiscuous crowd of people on a large area. Mr. R. Moreland, in a letter to *Engineering,* April 28th, 1905, contributed the following information:

"In 1900 my firm had a contract for the Manchester Racecourse for some large stands, and to gain information we put as many men as possible on our 10-ton weighbridge, and we found that we could get ninety men as close as possible into a space of 14 ft. by 8 ft., or 112 sq. ft., and they weighed 115 cwt. 1 qr., which would equal 1 cwt. per square foot. We then asked them to jump, and the load went up to 1½ cwt.; they then ran four abreast across the machine, but no excess was recorded over the 1½ cwt. per square foot."

Mr. Schneider.

This experiment is particularly valuable, as it includes the impact produced by jumping and running. The experiment indicates that the extreme effect, including impact produced by a crowd of people, packed so closely that jumping is just possible, is equivalent to a static load of about 170 lb. per sq. ft.

Let us consider an extreme case of loading in a public assembly room, and take 170 lb. per sq. ft. as the effect produced by a crowd of moving, stamping or jumping people—which, however, is in excess, as the same results could not be reproduced on a large area—and compare this case with the requirements of the specifications. The specifications, as revised, give 100 lb. per sq. ft., including impact. This would make, together with a dead load of 100 lb. per sq. ft., a total load of 200 lb. per sq. ft. If the effect of the live load should be 170 lb. per sq. ft. instead of 100, as given in the specifications, the total load would be increased from 200 to 270 lb. per sq. ft., thus increasing the unit strain from 16 000 to 21 600 lb., which strain is so well within the elastic limit as to be entirely safe even for such an improbable and extreme case.

A floor, designed in accordance with the writer's specifications, for a ballroom, assembly room, gymnasium, armory, etc., will be entirely safe for all possible contingencies which may arise in the legitimate use of such floors.

Mr. Macdonald mentions an extreme case of actual loading in a 13 by 16-ft. room of an office building, where pamphlets were stored, besides a 1 600-lb. safe, etc., which produced a total live load of 40 000 lb., or 200 lb. per sq. ft.

If this load were carried on four beams, each beam would carry 10 000 lb. uniform or 5 000 lb. concentrated load, as provided for in the writer's specifications.

If the load were carried on three beams only, we would have 13 300 lb. per beam, as against 10 000 lb., which, with the dead weight of the floor, 85 lb. per sq. ft., would increase the unit strain on the beams from 16 000 to about 21 000 lb. per sq. in., which would not only be entirely safe under this improbable assumption, but would be safe even for a greater load.

Mr. Blakeley thinks that a live load of 1 000 lb. per lin. ft. on floor girders in office buildings is too small, to which the writer takes exception. In order to prove his assertion, he cites a hypothetical case intending to prove the possibility of producing a superimposed load of 1 250 lb. per lin. ft. on a floor girder of an office building, if a partition should be located over the girder and 5 000-lb. safes be placed one on each side of the partition in the center of the girder.

Mr. Blakeley's case is not only improbable, but absolutely impossible, as he does not place the safes on each side of the partition,

Mr. Schneider. but on the girder and one on top of the other, and lets them rest on a knife-edge in the center of the girder, instead of figuring the load as distributed over the length of the safe, or, what is more correct, assuming each safe to be carried on two beams.

If the safes were placed in the most unfavorable position on each side of the partition, and with the usual spacing of the beams, they would produce a bending moment on the girder equivalent to a load of less than 900 lb. per lin. ft., but not 1 250 lb. as figured by Mr. Blakeley.

A floor girder designed in accordance with the writer's specifications, assuming the dead weight of the floor at 85 lb. per sq. ft., would carry, besides the safes, an additional live load of 50 lb. per sq. ft. of floor area without producing a fiber strain of more than 20 000 lb. per sq. in. An additional floor load of 75 lb. per sq. ft. would produce a fiber strain of less than 24 000 lb. per sq. in., which is yet well within the elastic limit, and absolutely safe if the details and connections are designed in accordance with good practice.

Mr. Clermont also mentions two very extreme cases of overloading on floors of office buildings, in one case a vault weighing 12 tons, and in the other a portion of a vault weighing 17 tons, had to be moved over the floor of an office building, yet these floors, figured only for a uniform live load of 150 lb., without provision for heavy concentrations, proved sufficient. As he does not give the length and spacing of the beams, it is impossible to make a comparison with a floor designed in accordance with the writer's specifications for loading.

In this case the beams would have to carry the same load, regardless of length of span, conforming to the provisions of the specifications in reference to concentrated loads, which are intended to cover just such contingencies.

Mr. Clermont endeavors to reconcile the great variations of live loads adopted in various building laws on the ground of their vastly different requirements. While the requirements may be different in different portions of the globe, inhabited by different races under different climatic conditions, they do not apply to buildings erected for the same purposes and in the same country.

Specifications applicable to New York City would probably not fit the conditions in the Philippine or Hawaiian Islands, nor in countries where provisions have to be made to resist seismic disturbances.

There is no good reason why floors in dwelling houses in Chicago should be figured for 40 lb. per sq. ft. and in Baltimore for 75 lb., nor why office buildings should carry 100 lb. per sq. ft. in Chicago and only 60 lb. in Milwaukee, and why a crowd of school children should weigh 150 lb. per sq. ft. in Boston and only 50 lb. in Milwaukee.

Mr. Schneider.

The fact is, all those values are simply guesswork of the wildest kind. The conditions in all large cities in the United States are practically alike, and some uniformity in the practice of building construction would be very desirable and would be as beneficial as uniform practice in other branches of engineering has proved to be.

Mr. Lowinson states that in his own practice he uses greater distributed loads than those adopted by the writer; these loads are supposed to cover all extreme cases. The load of 150 lb. per sq. ft. proposed by him does not cover extreme cases of short-span beams, and would be entirely out of proportion for girders and columns.

Providing for uniform loads only is adopting the practice in bridge building of more than 35 years ago, when the usual specifications for live load were a uniform load of 1 ton per lin. ft. of track. This proved so disastrous as to induce engineers to adopt more rational methods conforming more nearly to the actual conditions.

He states that stables and carriage-houses should be designed for automobile loads, and that he would design a private stable with the loading given by the writer, but for the carriage-house of the stable, where trucks might be stored, he would assume a greater load. This is contrary to the writer's recommendations, as trucks, carriages, automobiles and wagons of any kind produce a smaller distributed load than almost any commodity which is generally stored on the floor of a stable or carriage-house. Mr. Lowinson found some automobiles weighing 4 000 lb. with a concentrated load of 1 500 lb. on one wheel, while the writer specifies 5 000 lb. concentration. Supposing this automobile to occupy a space of 6 by 12 ft. or 72 sq. ft., this would make a distributed load of only 55.5 lb. per sq. ft. of floor area. Mr. Lowinson suggests 250 lb., which would be entirely out of reason. The specified uniform load of 70 lb. per sq. ft., therefore, is in excess of that which can be produced by automobiles or trucks of any kind. In the revised specifications a uniform load of 80 lb. per sq. ft. has been adopted to cover the loads produced by hay, feed, etc.

The most unfavorable position in which these concentrated wheel loads could be placed would be to put both heavy wheels on the same beam.

The load produced would be equivalent to a single concentrated load of less than 3 000 lb. for any length of span. But in order to provide for future possible increase in the weight of automobiles, the writer has revised his concentrated loading to 8 000 lb., which, with the automobiles weighed by Mr. Lowinson, would provide for a row of automobiles, the heaviest concentrations all on one beam, for spans up to about 36 ft., with a unit strain of 16 000 lb. per sq. in., and would therefore be good for an increase in the weights of automobiles of at least 50%; which would only increase the fiber strain to about 20 000 lb. per sq. in.

Mr. Schneider. In reference to loads on sidewalks, the concentrated load of 10 000 lb. will generally cover all extreme cases of loading. The writer has changed the distributed load to 300 lb. per sq. ft. in his revised specifications, which, however, will be found to make no material difference in the size of the beams.

Referring to Mr. Aus' objection that the writer's specifications, if adopted, would induce designers, for the sake of economy, to use long spans, the writer begs to state that this is his intention, as he considers longer spans with material concentrated in large masses in a smaller number of members productive of better results and more rigid structures, and in accordance with the best practice.

The argument is advanced by some of the discussors that office buildings may be used at some time for storage and manufacturing purposes. Rooms in office buildings designed in accordance with the writer's specifications are entirely safe for ordinary storage as well as for light manufacturing; but nobody would venture to transform an office building into a machine shop for heavy work, an ice plant, or a storage warehouse, without thorough investigation. In many cases a number of stories have been added to existing buildings, yet nobody has ever suggested that all buildings should be designed to carry an additional number of stories.

Paragraph 7.—Loads on Ordinary Roofs.—The writer has probably not been explicit enough in defining what is meant by an ordinary roof, but has qualified it in the revised specifications by including roofs up to 80 ft. span: They are roofs of sheds and manufacturing buildings which do not carry any floor or ceiling. The spans of such roofs generally range between 30 and 80 ft.

Mr. Smith and Mr. Ketchum are of the opinion that the dead weight of the roof should be calculated in each case. The writer believes that there should be certain limitations, and that such hair-splitting refinements are not only unnecessary but objectionable. A good rule to use in calculating roof trusses of the ordinary kind is to take the snow and wind load combined at 30 lb. per sq. ft. of horizontal projection for all slopes. This will fit all ordinary cases; if the roof is flat, that load is produced by snow alone; if the slope is greater than 45°, in which case the snow load need not be considered, it is all wind, and proportionately for intermediate slopes.

The variations of dead weight per square foot of roof trusses, purlins and bracing, for roofs of 30 to 80 ft. span, is small, and, as the dead load increases with a larger roof area, the live load may be reduced. For a smaller roof we will have more live load per square foot than for a larger one, which is good practice, and has been suggested and adopted by the writer for floor loads on buildings. In other words, the dead load of the roof trusses to be used in calculations should be limited.

Mr. Schneider.

The weights of the trusses, designed in accordance with the writer's specifications, including purlins and bracing, vary from 4 to 8 lb. per sq. ft. for spans from 30 to 80 ft., with distances between centers of trusses varying from 8 to 16 ft. The weight of roof covering of different kinds per square foot of sloping surface may be assumed as follows:

Corrugated steel	3 lb.	
Slate on 2-in. plank	15 lb.	Add 8 lb. for each additional inch in thickness of plank.
Tar and gravel on 2-in. plank	13 lb.	
Tar and gravel on 3-in. concrete	32 lb.	Add 8 lb. for each additional inch in thickness of concrete.
Slate on 3-in. booktile, T purlins	26 lb.	

For ordinary roofs, a good rule is to make the thickness of the roof planking ¼ in. for each foot of span, with a limit of 2 in., thus 2-in. plank may be used up to 8 ft. span, 3-in. plank up to 12 ft. span, and 4-in. plank up to 16 ft. span.

Concrete slabs, 3 in. thick, may be used for spans up to 8 ft., and 4-in. slabs up to 12 ft. span.

Mr. Shankland thinks the weights of skylights should be included in the loads on ordinary roofs. The weights of skylights, including the frame, vary from 4 to 8 lb. per sq. ft., and are generally covered by the weight of roof covering excepting corrugated sheeting. Occasional skylights in a roof, therefore, may be neglected. If the roof is all glass, the writer would use a total load of 45 lb. per sq. ft.

Paragraph 10.—Reduction of Live Loads on Columns.—The wording of this paragraph, as copied from the New York building law, seems to leave a doubt in many minds as to how it should be interpreted. The writer changed the wording of it in accordance with Mr. Seaman's suggestion, and hopes that this will convey a clearer idea of its meaning. Mr. Crowell has very ably discussed this question; the writer's intention was that his Rule *B* should be followed. This reduction of live loads on columns should not apply to warehouses or other buildings which are likely to receive the maximum loads on all floors at the same time.

Paragraph 12.—Wind Pressure.—Messrs. Worcester, Shankland and Clarke think that the specified wind pressure of 30 lb. per sq. ft. is excessive. The writer believes that it is better to err on the side of safety, more particularly as the proper provision to resist the wind strains requires very little extra material, in many cases only a more rational disposition of the same, and that a structure so designed will offer extra stability and rigidity. The specifications do not call for any special wind bracing, but require that the steel

Mr. Schneider. framework be designed to resist the specified wind pressure. This may be done, without special wind bracing, by designing the framework and connections to meet those conditions.

The writer does not agree with Mr. Stern, that the provisions for wind pressure are ambiguous, but thinks they are very explicit. It is distinctly stated that the framework shall be considered as an independent structure, without walls, partitions and floors; in that case the total exposed surface of all parts composing the metal framework shall be the wind surface. In case of a finished building the sides and ends or the vertical projection of the roof is the wind surface. If the building is of the office type, and the walls, floors and partitions are sufficient to resist the additional wind pressure produced by the increased exposed surface, no additional provision is necessary in the steel framework.

If the building is of the shed type, without transverse walls, then the steel framework has to resist the entire wind force on the total exposed surface of the finished building.

The case mentioned by Mr. Hewes, where a large printing press in the upper floor of a building caused undue vibrations, shows the necessity of effective bracing, not only to resist the wind force, but also vibrations, and impart that rigidity which should exist in all permanent structures.

Paragraph 13.—Foundations.—Mr. Worcester suggests increasing the permissible loads on foundations. On foundations for bridge piers, higher pressures are generally allowed, but, as a settlement in the foundations of a building is more injurious than in a bridge pier, the writer thinks it advisable to be conservative. A hard and fast rule for safe loads on foundations cannot be given, as it is impossible to classify the various soils accurately. Careful examinations should be made in all cases where the bearing capacity of the soil is not already known. The bearing power on rock is not given in the table, as that generally depends upon the material resting on the foundation.

Paragraph 14.—Masonry.—It is suggested by Messrs. Worcester and Shankland to increase the permissible pressure on hard-burned brick with Portland-cement mortar to 15 and 12 tons, respectively, and on Portland-cement concrete to 30 tons per sq. ft. Mr. Clarke suggests 35 tons on Portland-cement concrete. The writer has increased these values in his revised specifications, also all the other working pressures on masonry in proportion, but not as much as suggested, as it is deemed advisable to be conservative on account of the poor workmanship usually prevailing in building work, these suggestions being probably based on the best kind of work and material. As suggested by Mr. Stern, all reference to walls with lime mortar is omitted in the revised specifications. In

concrete walls, in the writer's opinion, only Portland cement should be used. Mr. Schneider.

Paragraph 15.—Pressure on Wall-Plates.—Mr. Post suggests, for the bearing of beams or girders on walls, making, for convenience, the size of the wall-plate the square of the depth of the beam, which is the usual practice and gives good results, but should not be incorporated in a specification.

Mr. Worcester suggests 200 lb. on brickwork and rubble masonry, 400 lb. on Portland-cement concrete and first-class sandstone, and 800 lb. on granite.

Paragraph 16.—Bearing Power of Piles.—This subject has been so ably discussed by Messrs. Howe, Goodrich, Stern, Crowell, Crehore, Smith and Darrach that much useful information may be obtained from these discussions. The writer agrees with most of the discussors, that it would be better to omit pile-driving formulas from general specifications, and that the bearing capacity of piles should be determined by test piles where positive information cannot be obtained in any other way. General specifications, however, should prescribe limits which should not be exceeded.

Paragraph 17,—Permissible Strains.—Mr. Ketchum suggests specifying the allowable shear on the net section of webs of plate girders. This used to be the practice in former years, but lately has been discarded as unsatisfactory and an unnecessary refinement.

Paragraph 19.—Permissible Compression Strains.—Mr. Worcester proposes to round off the unit strains, using an even 1 000 lb., which he claims is easier of application. This would be a good rule to follow in one's own practice. The specifications are supposed to give the upper limits only. The objection to his rule is that for long slender columns he uses higher unit strains than are consistent with good practice.

Mr. Seaman, on the other hand, desires more refinement than that given by the straight-line formula.

For timber columns, the straight-line formula has been adopted, as suggested by Mr. Ketchum and others.

Paragraphs 22, 23 and 24.—The wording of these paragraphs has been revised, and the three combined into one under the heading of "Combined Strains," covering the same provisions in more condensed form. Mr. Ketchum suggests Johnson's formula for combined strains as the most satisfactory yet proposed, which is also the writer's opinion, but he believes that formulas should be avoided as much as possible, as their proper place is in textbooks and not in specifications.

Paragraph 25.—Alternate Strains.—Mr. Seaman thinks the provision, that members subject to alternate strains "shall be proportioned for the strain giving the largest section" does not appear to

Mr. Schneider. be sufficient. Alternate strains, such as those in web members of railroad bridges, do not occur in structures covered by the writer's specifications. It is therefore unnecessary to make provision for them in members, but as it has been the writer's aim to produce strong connections, provision is made in the revised specifications for alternate strains in the connections.

Paragraph 29.—Plate Girders.—Mr. Seaman objects to having one-eighth of the cross-section of the web figured as available flange area, as an unnecessary refinement. Mr. Seaman probably has in mind the net section, in which case one-sixth would be correct, but not for the cross-section.

Paragraph 33.—Limiting Depth of Beams and Girders.—The writer does not share the opinions of Messrs. Clarke and Shankland, that the depth of one-twentieth of the span is excessive for rolled beams, as he has never come across a case yet where 3 in. in the depth of the floor would make any difference, or spoil an architect's decorations.

In cases, however, where the depth of the beams or girders is limited to one-fifteenth of the span, provision should be made for increased flange sections in shallower beams.

Paragraph 35.—Cast Iron.—In accordance with the suggestions of Messrs. Llewellyn, Stern and Worcester, the permissible unit strains in tension and shear are included in the revised specifications.

Paragraph 40.—Adjustable Members.—The writer does not agree with Mr. Shankland that adjustable members are preferable for wind bracing in building work. Adjustable members may be unavoidable in certain cases, and may be used to advantage in temporary structures. In permanent structures, stiff bracing is always preferable, as it imparts additional rigidity to the frame. The writer has not used adjustable members in any permanent structure for many years.

Paragraph 44.—Floor Beams.—Mr. Seaman is correct in assuming that the provision in this paragraph implies that shelf-angle supports alone are not sufficient. Preferably, all girders should be riveted direct to the columns, in the same way as recognized as good practice in bridge construction. If this cannot be done, and shelf-angles have to be resorted to, to support the girder, additional connections should be provided to insure rigidity. The writer, however, objects to his proposed web connection to Z-bar columns as impractical.

In reference to his objection to material more than $\frac{3}{4}$ in. thick, the writer would call attention to the fact that bed plates for column bases more than 1 in. thick are frequently used, and eye-bars from $1\frac{1}{2}$ to 2 in. thick, which is unquestionably good practice.

The specifications relating to material and workmanship, as stated before, are practically the same as recommended by the Committee on Steel Structures, of the American Railway Engineering and Maintenance of Way Association, as far as they were applicable to building construction. Mr. Schneider.

This will also answer Mr. Aus' question as to the writer's reason for departing from the former practice of requiring steel from 60 000 to 70 000 lb. ultimate strength: Because it is the latest and best practice, and recognized by engineers to be the most suitable material for structural purposes.

Mr. Crowell's remarks, regarding the treatment of steel to be embedded in concrete, are very appropriate, and his method is to be recommended in cases where such treatment is necessary.

In conclusion, the writer desires to express his appreciation, to those who have favored him with discussions on this subject, for their valuable suggestions, which have assisted him in revising his specifications. These specifications as revised will be found to embody all the suggestions, which, in accordance with his opinion, would improve the same, and are hereby respectfully submitted to the members of the American Society of Civil Engineers.

REVISED
GENERAL SPECIFICATIONS
FOR STRUCTURAL WORK OF BUILDINGS.

PART I.—DESIGN.

Loads.

1.—Dead Load.—The "dead" load in all structures shall consist of the weight of walls, floors, partitions, roofs and all other permanent construction and fixtures.

2.—In calculating the "dead" loads, the weights of the different materials shall be assumed as given in Table 1.

3.—The minimum weight of fire-proof floors to be assumed in designing the floor system shall be 75 lb. per sq. ft. For columns, the actual weight of floors shall be used.

4.—For office buildings 10 lb. per sq. ft. of floor area shall be added to the dead load of the floor for movable partitions.

5.—Live Load on Floors.—The following table gives the "live" load on floors, to be assumed for different classes of buildings. These loads consist of:

a.—A uniform load per square foot of floor area;
b.—A concentrated load which shall be applied to any point of the floor;
c.—A uniform load per linear foot for girders.

The maximum result is to be used in calculations.

The specified concentrated loads shall also apply to the floor construction between the beams for a length of 5 ft.

TABLE OF LIVE LOADS.

Classes of buildings.	Live Loads, in Pounds.		
	Distributed Load.	Concentrated Load.	Load per linear foot of girder.
Dwellings, hotels, apartment-houses, dormitories, hospitals	40	2 000	500
Office buildings, upper stories	50	5 000	1 000
Schoolrooms, theater galleries, churches	60	5 000	1 000
Ground floors of office buildings, corridors and stairs in public buildings	80	5 000	1 000
Assembly rooms, main floors of theaters, ballrooms, gymnasia, or any room likely to be used for drilling or dancing.	floor 100 columns 50	5 000	1 000
Ordinary stores and light manufacturing, stables and carriage-houses	80	8 000	1 000
Sidewalks in front of buildings	300	10 000	1 000
Warehouses and factories	from 120 up	Special.	Special.
Charging floors for foundries	" 300 "	"	"
Power-houses, for uncovered floors	" 200 "	The actual weights of engines, boilers, stacks, etc., shall be used, but in no case less than 200 lb. per sq. ft.	

6.—If heavy concentrations, like safes, armatures, or special machinery, are likely to occur on floors, provision should be made for them.

7.—Crane Loads and Impact.—For structures carrying traveling machinery, such as cranes, conveyors, etc., 25% shall be added to the strains resulting from such live load, to provide for the effects of impact and vibrations. (For crane loads, see Table 2.)

8.—Live Loads on Flat Roofs.—Flat roofs of office buildings, hotels, apartment-houses, etc., which can be loaded by crowds of people, shall be treated as floors, and the same distributed live loads shall be used as specified for hotels and dwelling-houses.

9.—Wind Pressure.—The wind pressure shall be assumed at 30 lb. per sq. ft., acting in any direction horizontally:

First.—On the sides and ends of buildings and on the actually exposed surface, or the vertical projection of roofs;

Second.—On the total exposed surfaces of all parts composing the metal framework. The framework shall be considered an independent structure, without walls, partitions or floors.

10.—Live Loads on Roofs.—Roofs shall be proportioned to carry in addition to their own weight the following live loads:

a.—A snow load, per horizontal square foot of roof, of 25 lb. for all slopes up to 20°, this load to be reduced 1 lb. for every degree of increase in the slope up to 45°, above which no snow load is considered.

b.—A wind load as specified in Paragraph 9.

The possibility of a partial snow load has to be considered.

The above loads given for snow are the minimum values for localities where snow is likely to occur. In severe climates these snow loads should be increased in accordance with the actual conditions existing in those localities. In tropical climates the snow loads may be neglected.

11.—Loads on Ordinary Roofs.—Ordinary roofs, up to 80 ft. span, shall be proportioned to carry the following minimum loads, per square foot of exposed surface, applied vertically, to provide for dead, wind and snow loads combined:

Gravel or Composition Roofing:	On boards, flat slope, 1 to 6, or less	50 lb.
	On boards, steep slope, more than 1 to 6	45 "
	On 3-in. flat tile or cinder concrete	60 "
Corrugated sheeting, on boards or purlins		40 "
Slate:	On boards or purlins	50 "
	On 3-in. flat tile or cinder concrete	65 "
Tile, on steel purlins		55 "

For roofs in climates where no snow is likely to occur, reduce the foregoing loads by 10 lb. per sq. ft., but no roof or any part thereof shall be designed for less than 40 lb. per sq. ft.

12.—Live Loads on Columns.—For columns, the specified uniform live loads per square foot shall be used, with a minimum of 20 000 lb. per column.

13.—Reduction of Live Load on Columns.—For columns carrying more than five floors, these live loads may be reduced as follows:

For columns supporting the roof and top floor, no reduction;

For columns supporting each succeeding floor, a reduction of 5% of the total live load may be made until 50% is reached, which reduced load shall be used for the columns supporting all remaining floors.

This reduction is not to apply to live load on columns of warehouses, and similar buildings which are likely to be fully loaded on all floors at the same time.

14.—Loads on Foundations.—The live loads on foundations shall be assumed to be the same as for the footings of columns. The

areas of the bases of the foundations shall be proportioned for the dead load only. That foundation which receives the largest ratio of live to dead load shall be selected and proportioned for the combined dead and live loads. The dead load on this foundation shall be divided by the area thus found, and this reduced pressure per square foot shall be the permissible working pressure to be used for the dead load of all foundations.

Unit Strains.

Substructure.

15.—Foundations.—Pressure on foundations not to exceed, in tons per square foot:

Soft clay	1
Ordinary clay and dry sand mixed with clay	2
Dry sand and dry clay	3
Hard clay and firm, coarse sand	4
Firm, coarse sand and gravel	6

16.—Masonry.—Working pressure in masonry not to exceed, in tons per square foot:

Common brick, Rosendale-cement mortar	10
" " Portland-cement mortar	12
Hard-burned brick, Portland-cement mortar	15
Rubble masonry, Rosendale-cement mortar	8
" " Portland-cement mortar	10
Coursed rubble, Portland-cement mortar	12
First-class masonry, sandstone	20
" " " limestone	25
" " " granite	30
Concrete for walls:	
Portland cement 1-2-5	20
" " 1-2-4	25

17.—Pressure on Wall-Plates.—The pressure of beams, girders, wall-plates, column bases, etc., on masonry shall not exceed the following, in pounds per square inch:

On brickwork with cement mortar	200
" rubble masonry with cement mortar	200
" Portland-cement concrete	350
" first-class sandstone	400
" " " limestone	500
" " " granite	600

18.—Bearing Power of Piles.—The maximum load carried by any pile shall not exceed 40 000 lb., or 600 lb. per sq. in. of its average cross-sections.

Piles driven in firm soil to rock may be loaded to the above limits. Piles driven through loose, wet soil to solid rock or equivalent bearing shall be figured as columns with a maximum unit strain of 600 lb. per sq. in., properly reduced.

The minimum distance between centers of piles shall be 2½ ft.

Superstructure.

Steel.

19.—Permissible Strains.—All parts of the structure shall be proportioned so that the sum of the dead and live loads, together with the impact, if any, shall not cause the strains to exceed those given in the following table:

	Pounds per square inch.
Tension, net section	16 000
Direct compression	16 000
Shear, on rivets and pins	12 000
Shear, on bolts and field rivets	9 000
Shear, on plate-girder web (gross section)	10 000
Bearing pressure, on pins and rivets	24 000
Bearing pressure, on bolts and field rivets	18 000
Fiber strain, on pins	24 000

20.—Permissible Compression Strains.—For compression members, the permissible strain of 16 000 lb. per sq. in. shall be reduced by the following formula:

$$p = 16\,000 - 70\,\frac{l}{r}$$

Where p = permissible working strain per square inch in compression;

l = length of piece, in inches, from center to center of connections;

r = least radius of gyration of the section, in inches.

21.—For wind bracing, and the combined strains due to wind and the other loading, the permissible working strains may be increased 25%, or to 20 000 lb. for direct compression or tension.

22.—Provision for Eccentric Loading.—In proportioning columns, provision must be made for eccentric loading.

23.—Expansion Rollers.—The pressure per linear inch on expansion rollers shall not exceed 600 d, where d = diameter of rollers, in inches.

24.—Combined Strains.—Members subject to the action of both axial and bending strains shall be proportioned so that the greatest fiber strain will not exceed the allowed limits for the axial tension or compression in that member.

25.—Reversal of Strains.—Members subject to reversal of strains shall be proportioned for the strain giving the largest section, but their connections shall be proportioned for the sum of the strains.

26.—Net Sections.—Net sections must be used in calculating tension members, and, in deducting the rivet holes; they must be taken $\frac{1}{8}$ in. larger than the nominal size of the rivets.

27.—Pin-connected riveted tension members shall have a net section through the pin holes 25% in excess of the net section of the body of the member. The net section back of the pin hole shall be at least 0.75 of the net section through the pin hole.

28.—Compression Members Limiting Length.—No compression member shall have a length exceeding 125 times its least radius of gyration, except those for wind and lateral bracing, which may have a length not exceeding 150 times the least radius of gyration.

29.—Plate Girders.—Plate girders shall be proportioned on the assumption that one-eighth of the gross area of the web is available as flange area. The compression flange shall have at least the same sectional area as the tension flange, but the unsupported length of the flange shall not exceed 16 times its width.

30.—In plate girders used as crane runways, if the unsupported length of the compression flange exceeds 12 times its width, the flange shall be figured as a column between the points of support.

31.—Web Stiffeners.—The web shall have stiffeners at the ends and inner edges of bearing plates, and at all points of concentrated loads, and also at intermediate points, when the thickness of the web is less than one-sixtieth of the unsupported distance between flange angles, generally not farther apart than the depth of the full web plate, with a minimum limit of 5 ft.

32.—Rolled Beams.—I-beams, and channels used as beams or girders, shall be proportioned by their moments of inertia.

33.—Limiting Depth of Beams and Girders.—The depth of rolled beams in floors shall be not less than one-twentieth of the span, and, if used as roof purlins, not less than one-thirtieth of the span.

In case of floors subject to shocks and vibrations, the depth of beams and girders shall be limited to one-fifteenth of the span. If shallower beams are used, the sectional area shall be increased until the maximum deflection is not greater than that of a beam having a depth of one-fifteenth of the span, but the depth of such beams shall in no case be less than one-twentieth of the span.

Cast Iron.

34.—Permissible Strains.—Compression. . . .12 000 lb. per sq. in.
Tension . . . . 2 500 " " " "
Shear . . . . 1 500 " " " "

Timber.

35.—Timber.—The timber parts of the structure shall be proportioned in accordance with the following unit strains, given in pounds per square inch:

Kind of timber.	Transverse loading.	End bearing.	Columns under 10 diameters.	Bearing across fiber.	Shear along fiber.
White Oak	1 200	1 200	1 000	500	200
Long-Leaf Yellow Pine	1 500	1 500	1 000	350	100
White Pine and Spruce	1 000	1 000	600	200	100
Hemlock	800	800	500	200	100

Columns may be used with a length not exceeding 45 times the least dimension. The unit strain for lengths of more than 10 times the least dimension shall be reduced by the following formula:

$$p = C - \frac{C}{100} \frac{l}{d}$$

Where C equals unit strains, as given above for short columns;
l " length of column, in inches;
d " least side of column, in inches.

36.—Planking.—For the thickness of floor and roof planking, see Table 3.

DETAILS OF CONSTRUCTION.

37.—Minimum Thickness of Material.—No steel of less than ¼ in. thickness shall be used, except for lining or filling vacant spaces.

38.—Adjustable Members.—Adjustable members in any part of structures shall preferably be avoided.

39.—Symmetrical Sections.—Sections shall preferably be made symmetrical.

40.—Connections.—The strength of connections shall be sufficient to develop the full strength of the member.

41.—No connection, except lattice bars, shall have less than two rivets.

42.—Floor Beams.—Floor beams shall generally be rolled steel beams.

43.—For fire-proof floors, they shall generally be tied with tie-

rods at intervals not exceeding eight times the depth of the beams. This spacing may be increased for floors which are not of the arch type of construction. Holes for tie-rods, where the construction of the floor permits, shall be spaced about 3 in. above the bottom of the beam.

44.—Beam Girder.—When more than one rolled beam is used to form a girder, they shall be connected by bolts and separators at intervals of not more than 5 ft. All beams having a depth of 12 in. and more shall have at least two bolts to each separator.

45.—Wall Ends of Beams and Girders.—Wall ends of a sufficient number of joists and girders shall be anchored securely to impart rigidity to the structure.

46.—Wall-Plates and Column Bases.—Wall-plates and column bases shall be constructed so that the load will be well distributed over the entire bearing. If they do not get the full bearing on the masonry, the deficiency shall be made good with Portland-cement mortar.

47.—Floor Girders.—The floor girders may be rolled beams or plate girders; they shall preferably be riveted or bolted to columns by means of connection angles. Shelf angles or other supports may be provided for convenience during erection.

48.—Flange Plates.—The flange plates of all girders shall be limited in width, so as not to extend, beyond the outer line of rivets connecting them to the angles, more than 6 in., or more than eight times the thickness of the thinnest plate.

49.—Web Stiffeners.—Web stiffeners shall be in pairs, and shall have a close bearing against the flange angles. Those over the end bearing, or forming the connection between girder and column, shall be on fillers. Intermediate stiffeners may be on fillers or crimped over the flange angles. The rivet pitch in stiffeners shall not be more than 5 in.

50.—Web Splices.—Web plates of girders must be spliced at all points by a plate on each side of the web, capable of transmitting the full strain through splice rivets.

51.—Columns.—Columns shall be designed so as to provide for effective connections of floor beams, girders or brackets.

They shall preferably be continuous over several stories.

52.—Column Splices.—The splices shall be strong enough to resist the bending strain and make the columns practically continuous for their whole length.

53.—Trusses.—Trusses shall preferably be riveted structures. Heavy trusses, of long span, where the riveted field connections would become unwieldy, or for other good reasons, may be designed as pin-connected structures.

54.—Intersecting Members.—Main members of trusses shall be designed so that the neutral axes of intersecting members shall meet in a common point.

55.—Roof Trusses.—Roof trusses shall be braced in pairs in the plane of the chords.

Purlins shall be made of shapes, or riveted-up plate, or lattice girders.

Trussed purlins will not be allowed.

56.—Eye-Bars.—The eye-bars in pin-connected trusses composing a member shall be as nearly parallel to the axis of the truss as possible.

57.—Spacing of Rivets.—The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than 3 in. for $\frac{7}{8}$-in. rivets, $2\frac{1}{2}$ in. for $\frac{3}{4}$-in. rivets, $2\frac{1}{4}$ in. for $\frac{5}{8}$-in. rivets, and $1\frac{3}{4}$ in. for $\frac{1}{2}$-in. rivets.

58.—For angles with two gauge lines, with rivets staggered, the maximum in each line shall be twice as great as given in Paragraph 57, and, where two or more plates are used in contact, rivets not more than 12 in. apart in any direction shall be used to hold the plates together.

59.—The pitch of the rivet, in the direction of the strain, shall not exceed 6 in., nor 16 times the thinnest outside plate connected, and not more than 50 times that thickness at right angles to the strain.

60.—Edge Distance.—The minimum distance from the center of any rivet hole to a sheared edge shall be $1\frac{1}{2}$ in. for $\frac{7}{8}$-in. rivets, $1\frac{1}{4}$ in. for $\frac{3}{4}$-in. rivets, $1\frac{1}{8}$ in. for $\frac{5}{8}$-in. rivets, and 1 in. for $\frac{1}{2}$-in. rivets; and to a rolled edge, $1\frac{1}{4}$, $1\frac{1}{8}$, 1 and $\frac{7}{8}$ in., respectively.

61.—The maximum distance from any edge shall be eight times the thickness of the plate.

62.—Maximum Diameter.—The diameter of the rivets in any angle carrying calculated strains shall not exceed one-quarter of the width of the leg in which they are driven. In minor parts, rivets may be $\frac{1}{8}$ in. greater in diameter.

63.—Pitch at Ends.—The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets for a length equal to one and one-half times the maximum width of the member.

64.—Tie-Plates.—The open sides of compression members shall be provided with lattice having tie-plates at each end and at intermediate points where the lattice is interrupted. The tie-plates shall be as near the ends as practicable. In main members, carrying calculated strains, the end tie-plates shall have a length not less than the distance between the lines of rivets connecting them to the flanges, and intermediate ones not less than half this distance.

Their thickness shall be not less than one-fiftieth of the same distance.

65.—Lattice.—The thickness of lattice bars shall be not less than one-fortieth for single lattice and one-sixtieth for double lattice, of the distance between end rivets; their minimum width shall be as follows:

For 15-in. channels, or built sections with $3\frac{1}{2}$ and 4-in. angles.....	$2\frac{1}{2}$ in. ($\frac{7}{8}$-in. rivets);
For 12, 10 and 9-in. channels, or built sections with 3-in. angles....	$2\frac{1}{4}$ in. ($\frac{3}{4}$-in. rivets);
For 8 and 7-in. channels, or built sections with $2\frac{1}{2}$-in. angles...	2 in. ($\frac{5}{8}$-in. rivets);
For 6 and 5-in. channels, or built sections with 2-in. angles....	$1\frac{3}{4}$ in. ($\frac{1}{2}$-in. rivets).

66.—Lattice bars with two rivets shall generally be used in flanges more than 5 in. wide.

67.—Angle of Lattice.—The inclination of lattice bars with the axis of the member, generally, shall be not less than 45°, and when the distance between the rivet lines in the flange is more than 15 in., if a single rivet bar is used, the lattice shall be double and riveted at the intersection.

68.—Spacing of Lattice.—The pitch of lattice connections, along the flange divided by the least radius of gyration of the member between connections, shall be less than the corresponding ratio of the member as a whole.

69.—Faced Joints.—Abutting joints in compression members when faced for bearing shall be spliced sufficiently to hold the connecting members accurately in place.

70.—All other joints in riveted work, whether in tension or compression, shall be fully spliced.

71.—Pin Plates.—Pin holes shall be reinforced by plates where necessary; and at least one plate shall be as wide as the flange will allow; where angles are used, this plate shall be on the same side as the angles. The plates shall contain sufficient rivets to distribute their portion of the pin pressure to the full cross-section of the member.

72.—Pins.—Pins shall be long enough to insure a full bearing of all parts connected upon the turned-down body of the pin.

73.—Members packed on pins shall be held against lateral movement.

74.—Bolts.—Where members are connected by bolts, the body of these bolts shall be long enough to extend through the metal. A washer at least $\frac{3}{16}$ in. thick shall be used under the nut.

75.—Fillers.—Fillers between parts carrying strain shall have a sufficient number of independent rivets to transmit the strain to the member to which the filler is attached.

76.—Temperature.—Provision shall be made for expansion and contraction, corresponding to a variation of temperature of 150° fahr., where necessary.

77.—Rollers.—Expansion rollers shall be not less than 4 in. in diameter.

78.—Stone Bolts.—Stone bolts shall extend not less than 4 in. into granite pedestals and 8 in. into other material.

79.—Anchorage.—Columns which are strained in tension at their base shall be anchored to the foundations.

80.—Anchor bolts shall be long enough to engage a mass of masonry, the weight of which shall be one and one-half times the tension in the anchor.

81.—Bracing.—Lateral, longitudinal and transverse bracing in all structures shall preferably be composed of rigid members.

TABLE 1.—WEIGHTS OF BUILDING MATERIALS, ETC., IN POUNDS PER CUBIC FOOT.

MATERIAL.	WEIGHT.	MATERIAL.	WEIGHT.
Paving brick	150	Plaster of paris	140
Common building brick	120	Glass	160
Soft building brick	100	Snow, freshly fallen	10
Granite	170	Snow, wet	50
Marble	170	Spruce	25
Limestone	160	Hemlock	25
Sandstone	145	White pine	25
Slag	40	Douglas fir	30
Gravel	120	Yellow pine	40
Slate	175	White oak	50
Sand, clay and earth (dry)	100	Common brickwork	100-120
Sand, clay and earth (wet)	120	Rubble masonry	130-150
Mortar	100	Ashlar masonry	140-160
Stone concrete	130-150	Cast iron	450
Cinder concrete	70	Wrought iron	480
Paving asphaltum	100	Steel	490

Plaster, ceiling, 10 to 15 lb. per sq. ft.

TABLE 2.—Typical Electric Traveling Cranes.

Capacity, in tons.	Span.	Wheel base.	Maximum wheel load, in pounds.	s.	v.	Weight of Rail for: Plate girders.	Weight of Rail for: Beams.
5	40	8 ft. 6 in.	12 000	10 in.	7 ft.	40 lb. per yd.	40
	60	9 " 0 "	13 000	"	"	40 "	40
10	40	9 " 0 "	19 000	"	"	45 "	40
	60	9 " 6 "	21 000	"	"	45 "	40
15	40	9 " 6 "	26 000	"	"	50 "	50
	60	10 " 0 "	29 000	"	"	50 "	50
20	40	10 " 0 "	33 000	12 in.	8 ft.	55 "	50
	60	10 " 6 "	36 000	"	"	55 "	50
25	40	10 " 0 "	40 000	"	"	60 "	50
	60	10 " 6 "	44 000	"	"	60 "	50
30	40	10 " 6 "	48 000	"	"	70 "	60
	60	11 " 0 "	52 000	"	"	70 "	60
40	40	11 " 0 "	64 000	14 in.	9 ft.	80 "	60
	60	12 " 0 "	70 000	"	"	80 "	60
50	40	11 " 0 "	72 000	"	"	100 "	60
	60	12 " 0 "	80 000	"	"	100 "	60

1.—Wheel-load can be assumed as distributed in top flange, over a distance equal to depth of girder, with a maximum limit of 30 in.

2.—In addition to the vertical load, the top flanges of the girder shall withstand a lateral loading of two-tenths of the lifting capacity of the crane, equally divided between the four wheels of the crane.

s = side clearance from center of rail.

v = vertical clearance from top of rail.

TABLE 3.—Thickness of Spruce and White Pine Plank for Floors.

Thickness, in Inches, for Various Loads per Square Foot of Plank.

Span, in feet.	lb. 30	lb. 40	lb. 50	lb. 75	lb. 100	lb. 125	lb. 150	lb. 175	lb. 200	lb. 225	lb. 250	lb. 275	lb. 300	lb. 325	lb. 350	lb. 375	lb. 400
4	0.9	1.1	1.2	1.5	1.7	1.9	2.1	2.2	2.4	2.5	2.7	2.8	2.9	3.1	3.2	3.3	3.4
5	1.2	1.4	1.5	1.9	2.1	2.4	2.6	2.8	3.0	3.2	3.4	3.5	3.7	3.8	4.0	4.1	4.3
6	1.4	1.6	1.8	2.2	2.6	2.9	3.1	3.4	3.6	3.8	4.0	4.2	4.4	4.6	4.8	4.9	5.1
7	1.7	1.9	2.1	2.6	3.0	3.3	3.7	3.9	4.2	4.5	4.7	4.9	5.2	5.4	5.6	5.8	6.0
8	1.9	2.2	2.4	3.0	3.4	3.8	4.4	4.5	4.8	5.1	5.4	5.7	5.9	6.1	....	....	....
9	2.1	2.5	2.7	3.4	3.9	4.3	4.7	5.1	5.4	5.8	6.1	....	....	....	....	....	....
10	2.4	2.7	3.1	3.7	4.3	4.8	5.2	5.6	6.0	....	....	....	....	....	....	....	....
11	2.6	3.0	3.4	4.1	4.7	5.3	5.8	....	....	....	....	....	....	....	....	....	....
12	2.9	3.3	3.7	4.5	5.2	....	....	....	....	....	....	....	....	....	....	....	....
13	3.1	3.6	4.0	4.9	5.6	....	....	....	....	....	....	....	....	....	....	....	....
14	3.4	3.9	4.3	5.3	6.1	....	....	....	....	....	....	....	....	....	....	....	....

For yellow pine use nine-tenths of the above thicknesses.

PART II.—MATERIAL AND WORKMANSHIP.

MATERIAL.

1.—Steel.—All parts of the structure shall be of rolled steel, except column bases, bearing plates or minor details, which may be of cast iron or cast steel.

2.—Process of Manufacture.—Steel may be made by the open-hearth or by the Bessemer process.

3.—The chemical and physical properties shall conform to the following limits:

Chemical and Physical Properties.	Structural steel.	Rivet steel.	Steel castings.
Phosphorus, maximum, Basic	0.04%	0.04%	0.05%
Phosphorus, maximum, Acid	0.08%	0.04%	0.08%
Sulphur, maximum	0.05%	0.04%	0.05%
Ultimate tensile strength; pounds per square inch	Desired 60 000	Desired 50 000	Not less than 65 000
Elongation: minimum percentage in 8 in.	$\frac{1\ 500\ 000^{*}}{\text{Ultimate tensile strength.}}$	$\frac{1\ 500\ 000}{\text{Ultimate tensile strength.}}$	....
Elongation: minimum percentage in 2 in.	22	....	18
Character of fracture	Silky.	Silky.	Silky or fine granular.
Cold bends without fracture	180° flat.†	180° flat.§	90°

* See Paragraph 18. † See Paragraphs 15 and 16. § See Paragraph 17.

4.—The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

5.—Allowable Variations.—Tensile tests of steel showing an ultimate strength within 5 000 lb. of that desired will be considered satisfactory.

6.—Chemical Analyses.—Chemical determinations of the precentages of carbon, phosphorus, sulphur and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector.

7.—Form of Specimens for Plates, Shapes and Bars.—Specimens for tensile and bending tests, for plates, shapes and bars, shall be made by cutting coupons from the finished product, which shall have both faces rolled and both edges milled to the form shown by Fig. 1; or with both edges parallel; or they may be turned to a diameter of $\frac{3}{4}$ in. for a length of at least 9 in., with enlarged ends.

8.—Rivets.—Rivet rods shall be tested as rolled.

9.—Pins and Rollers.—Specimens shall be cut from the finished rolled or forged bar, in such manner that the center of the specimen

shall be 1 in. from the surface of the bar. The specimen for the tensile test shall be turned to the form shown by Fig. 2. The specimen for the bending test shall be 1 in. by ½ in. in section.

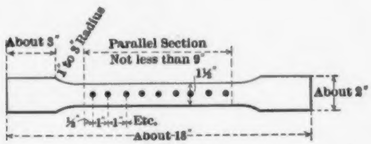


FIG. 1.

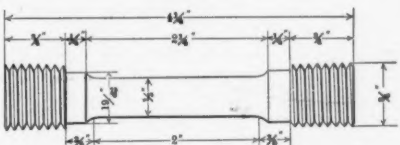


FIG. 2.

10.—Steel Castings.—The number of tests will depend on the character and importance of the castings. Specimens shall be cut cold from coupons moulded and cast on some portion of one or more castings from each melt, or from the sink-heads, if the heads are of sufficient size. The coupon or sink-head, so used, shall be annealed with the casting before it is cut off. Test specimens shall be of the form prescribed for pins and rollers.

11.—Annealed Specimens.—Material which is to be used without annealing or further treatment shall be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen for tensile tests representing such material shall be cut from properly annealed or similarly treated short lengths of the full section of the bar.

12.—Number of Tests.—At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing ⅜ in. and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

13.—Modifications in Elongation.—For material more than ¾ in. in thickness, for each ⅛ in. in thickness above ¾ in., a deduction of 1% will be allowed from the specified elongation.

14.—Bending Tests.—Bending tests may be made by pressure or by blows. Plates, shapes and bars less than 1 in. thick shall bend as called for in Paragraph 3.

15.—Thick Material.—Full-sized material, for eye-bars and other steel 1 in. or more in thickness, tested or rolled, shall bend cold 180°

around a pin the diameter of which is equal to twice the thickness of the bar, without fracture on the outside of the bend.

16.—Bending Angles.—Angles ¾ in. and less in thickness shall open flat, and angles ½ in. and less in thickness shall bend shut, cold, under blows of a hammer, without sign of fracture. This test will be made only when required by the inspector.

17.—Nicked Bends.—Rivet steel, when nicked and bent around a bar of the same diameter as the rivet rod, shall give a gradual break and a fine, silky, uniform fracture.

18.—Finish.—Finished material shall be free from injurious seams, flaws, cracks, defective edges, or other defects, and shall have a smooth, uniform, workmanlike finish. Plates 36 in. and less in width shall have rolled edges.

19.—Stamping.—Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it. Steel for pins and rollers shall be stamped on the end. Rivet and lattice steel and other small parts may be bundled with the above marks on an attached tag.

20.—Defective Material.—Material which, subsequent to the foregoing tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious defects, will be rejected at the shop, and shall be replaced by the manufacturer at his own cost.

21.—Allowable Variation in Weight.—A variation in cross-section or weight in the finished members of more than 2½% from that specified will be sufficient cause for rejection.

22.—Cast Iron.—Iron castings shall be made of tough, gray iron, free from injurious cold-shuts or blow-holes, true to pattern and of workmanlike finish. Test pieces 1 in. square shall be capable of sustaining on a clear span of 12 in. a central load of at least 2 500 lb., and deflect at least 0.15 in. before rupture.

Workmanship.

23.—General.—All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works.

24.—Straightening Material.—Material shall be thoroughly straightened in the shop, by methods which will not injure it, before being laid off or worked in any way.

25.—Finish.—Shearing shall be done neatly and accurately, and all portions of the work exposed to view shall be neatly finished.

26.—Rivets.—The size of rivets called for on the plans shall be understood to mean the actual size of the cold rivet before heating.

27.—Rivet holes.—The diameter of the punch for material not more than $\frac{5}{8}$ in. thick shall be not more than $\frac{1}{16}$ in., nor that of the die more than $\frac{1}{8}$ in. larger than the diameter of the rivet. Material more than $\frac{3}{4}$ in. thick, excepting in minor details, shall be subpunched and reamed or drilled from the solid.

28.—Punching.—Punching shall be done accurately. Slight inaccuracy in the matching of holes may be corrected with reamers. Drifting to enlarge unfair holes will not be allowed. Poor matching of holes will be cause for rejection, at the option of the inspector.

29.—Assembling.—Riveted members shall have all parts well pinned up and firmly drawn together with bolts before riveting is commenced. Contact surfaces shall be painted. (See Paragraph 52.)

30.—Lattice Bars.—Lattice bars shall have neatly rounded ends, unless otherwise called for.

31.—Web Stiffeners.—Stiffeners shall fit neatly between the flanges of girders. Where tight fits are called for, the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

32.—Splice Plates and Fillers.—Web splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$ in. of flange angles.

33.—Connection Angles.—Connection angles for floor girders shall be flush with each other and correct as to position and length of girder.

34.—Riveting.—Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

35.—Rivets.—Rivets shall look neat and finished, with heads of approved shape, full, and of equal size. They shall be central on the shank and shall grip the assembled pieces firmly. Recupping and caulking will not be allowed. Loose, burned, or otherwise defective rivets shall be cut out and replaced. In cutting out rivets, great care shall be taken not to injure the adjoining metal. If necessary, they shall be drilled out.

36.—Field Bolts.—Wherever bolts are used in place of rivets which transmit shear, such bolts must have a driving fit. A washer not less than $\frac{1}{4}$ in. thick shall be used under the nut.

37.—Members to be Straight.—The several pieces forming one built member shall be straight and shall fit closely together, and finished members shall be free from twists, bends or open joints.

38.—Finish of Joints.—Abutting joints shall be cut or dressed true and straight and fitted closely together, especially where open to view. In compression joints depending on contact bearing, the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

39.—Eye-Bars.—Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of the heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to break in the body with a silky fracture, when tested to rupture. The thickness of the head and neck shall not vary more than $\frac{1}{16}$ in. from the thickness of the bar.

40.—Boring Eye-Bars.—Before boring, each eye-bar shall be perfectly annealed and carefully straightened. Pin holes shall be in the center line of bars and in the center of heads. Bars of the same length shall be bored so accurately that, when placed together, pins $\frac{1}{32}$ in. smaller in diameter than the pin holes can be passed through the holes at both ends of the bars at the same time.

41.—Pin Holes.—Pin holes shall be bored true to gauges, smooth and straight; at right angles to the axis of the member, and parallel to each other, unless otherwise called for. Wherever possible, the boring shall be done after the member is riveted up.

42.—Variation in Pin Holes.—The distance from center to center of pin holes shall be correct within $\frac{1}{32}$ in., and the diameter of the hole not more than $\frac{1}{50}$ in. larger than that of the pin, for pins up to 5 in. diameter, and $\frac{1}{32}$ in. for larger pins.

43.—Pins and Rollers.—Pins and rollers shall be turned accurately to gauges, and shall be straight, smooth and entirely free from flaws.

44.—Pilot Nuts.—At least one pilot and driving nut shall be furnished for each size of pin for each structure.

45.—Screw Threads.—Screw threads shall make tight fits in the nuts, and shall be United States standard, except for diameters greater than $1\frac{3}{8}$ in., when they shall be made with six threads per inch.

46.—Annealing.—Steel, except in minor details, which has been partially heated shall be properly annealed.

47.—Steel Castings.—All steel castings shall be annealed.

48.—Welds.—Welds in steel will not be allowed.

49.—Bed-Plates.—Expansion bed-plates shall be planed true and smooth. Cast wall-plates shall be planed at top and bottom. The cut of the planing tool shall correspond with the direction of expansion.

50.—Shipping Details.—Pins, nuts, bolts, rivets and other small details shall be boxed or crated.

51.—Weight.—The weight of every piece and box shall be marked on it in plain figures.

Painting.

52.—*Shop Painting.*—Steelwork, before leaving the shop, shall be thoroughly cleaned and given one good coating of pure linseed oil, or such paint as may be called for, well worked into all joints and open spaces.

53.—In riveted work, the surfaces coming in contact shall be painted before being riveted together.

54.—Pieces and parts which are not accessible for painting after erection, shall have two coats of paint before leaving the shop.

55.—Steelwork to be entirely embedded in concrete shall not be painted.

56.—Painting shall be done only when the surface of the metal is perfectly dry. It shall not be done in wet or freezing weather, unless protected under cover.

57.—Machine-finished surfaces shall be coated with white lead and tallow before shipment, or before being put out into the open air.

58.—*Field Painting.*—After the structure is erected, the metalwork shall be painted thoroughly and evenly with an additional coat of paint, mixed with pure linseed oil, of such quality and color as may be selected. The field paint shall be of different color from the shop paint.

Inspection and Testing.

59.—The manufacturer shall furnish all facilities for inspecting and testing the weight, quality of material and workmanship. He shall furnish a suitable testing machine for testing the specimens, as well as prepare the pieces for the machine, free of cost.

60.—When an inspector is furnished by the purchaser, he shall have full access at all times to all parts of the works where material under his inspection is manufactured.

61.—The purchaser shall be furnished with complete copies of mill orders, and no material shall be rolled and no work done before he has been notified as to where the orders have been placed, so that he may arrange for the inspection.

62.—The purchaser shall also be furnished with complete shop plans, and must be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect the material and workmanship.

63.—Complete copies of shipping invoices shall be furnished to the purchaser with each shipment.

64.—If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the purchaser.

Full-Sized Tests.

65.—Full-sized parts of the structure may be tested at the option of the purchaser. Such tests on eye-bars and similar members, to prove the workmanship, shall be made at the manufacturer's expense, and shall be paid for by the purchaser, at contract price, if the tests are satisfactory. If the tests are not satisfactory, the members represented by them will be rejected. The expense of testing members, to prove their design, shall be paid for by the purchaser.

66.—In eye-bar tests, the ultimate strength, the elastic limit and the elongation in 10 ft., unless a different length is called for, shall be recorded.

MEMOIRS OF DECEASED MEMBERS.

ALPHONSE FTELEY, Past-President, Am. Soc. C. E.*

DIED JUNE 11TH, 1903.

Alphonse Fteley was born in Paris on April 10th, 1837, was educated in France, took the academic degree from the University of France, and, later, was graduated from L'École Polytechnique in 1859. From 1881 to 1884 he was a member of the Committee of Visitors to the Lawrence Scientific School of Harvard University, and, in 1898, he received from Columbia University the degree of M. A.

He began his professional life in France in 1859, and continued there in the general practice of his profession until 1865, when he came to the United States. He located in New York City, and in 1866 entered the office of the late William E. Worthen, Past-President, Am. Soc. C. E., as his general assistant, remaining with him until 1870. Then he opened an office, at 63 Bleecker Street, for general practice, in which he continued until 1873, when he was engaged by the Boston Water Board, under the City Engineer, Joseph P. Davis, M. Am. Soc. C. E., to be Resident Engineer in charge of the construction of the Sudbury River water-works system for the City of Boston. This work, costing about $5 000 000, was at that time considered as among those of the first importance in this country. It comprised several reservoirs, dams, etc., an aqueduct, 15.9 miles long, crossing two valleys on stone arch bridges, and was very successfully carried out. In connection with this work, Mr. Fteley carried out a series of observations for gauging the flow of the Sudbury River. These observations were conducted with great care and skill. With his assistant, F. P. Stearns, M. Am. Soc. C. E., he also conducted a series of experiments on the flow of water over weirs.

In 1880, upon the election of Assistant City Engineer Henry M. Wightman as City Engineer of Boston, Mr. Fteley was appointed Assistant City Engineer, and was engaged in designing many important structures, especially for the Water-Works, Main Drainage and Park Systems of the city, until 1884 when he left the service of the City of Boston to accept, from the Aqueduct Commission of New York City, the position of Principal Assistant Engineer on the new Croton Aqueduct, under Mr. Benjamin S. Church, the Chief Engineer.

*Memoir prepared by the following Committee: William Jackson, Joseph P. Davis, Charles S. Gowen and Charles Warren Hunt, Members, Am. Soc. C. E.

In 1886 his title was changed to Consulting Engineer, and, in 1888, upon the retirement of Mr. Church, he was appointed Chief Engineer. As Principal Assistant Engineer to Mr. Church, he had charge of the design of the aqueduct, of a dam for the Quaker Bridge site and of other important dams, structures and reservoirs and also of the preparation of the specifications and contracts for their construction. After his appointment as Chief Engineer investigations were renewed for the location and design of the proposed dam to complete the storage of the Croton water-shed, and it was decided to build the New Croton Dam at a point some distance up stream from the Quaker Bridge site.

He was exceptionally qualified by his experience to design and to direct the construction of works of this class, and was, without doubt, one of the best authorities of his time. Mr. Fteley continued his connection with this work until 1899 when he was compelled by ill health to resign and retire from the active practice of his profession. With the exception of the New Croton Dam and the Jerome Park Reservoir, all the important features of the Croton water supply were completed before his retirement.

Mr. Fteley was also engaged in an advisory capacity, and was consulted on many important engineering works. Mention may be made of the following: The works and studies under the charge of the Metropolitan Water Board of Massachusetts; of the Sewerage Commission of New Jersey; of the Rapid Transit Commission of New York City; of the Boston Rapid Transit Commission of 1892; of the Cataract Construction Company, of Niagara Falls, New York; of the Schuylkill Navigation Company; of the Board of Works of Newark, New Jersey; the Aqueduct Tunnel repairs of Washington, D. C.; the additional water supply of Rochester, New York; the supply of water to New York City from New Jersey; the additional water supply for Cincinnati, Ohio; and the additional supply for Brooklyn, New York. He was also a member of the Comité Technique of the New French Panama Canal Company.

Mr. Fteley was elected a Member of the American Society of Civil Engineers on January 5th, 1876, and in 1879 read a paper before it entitled "The Flow of the Sudbury River, Massachusetts, for the Years 1875 to 1879."* Three years later, in collaboration with his asistant, F. P. Stearns, M. Am. Soc. C. E., he presented what is perhaps his most valuable contribution to the science of hydraulics, a paper entitled "Description of Some Experiments on the Flow of Water Made during the Construction of Works for Conveying the Water of Sudbury River to Boston."† This paper received the award of the Norman Medal for 1882. He was active in the management of the Society for a number of years, serving as Director

* *Transactions*, Am. Soc. C. E., Vol. X, p. 225.

† *Transactions*, Am. Soc. C E., Vol. XII, p. 1.

in 1888 and as Vice-President in 1889, 1890 and 1891. In 1898 he was elected President, and, from the expiration of his term to the time of his death, continued to take great interest in the affairs of the Society.

In 1874 Mr. Fteley was elected a member of the Boston Society of Civil Engineers, and of the New England Water Works Association.

Mr. Fteley married in 1869, in New York City, Elise Susanne Maurier, widow of Jules Breuchaud. He left a daughter and four step-children.

An obscure heart trouble from which he suffered during the last thirty years of his life, and which caused two severe illnesses of prolonged duration in 1879 and 1886, limited Mr. Fteley's capacity for work at times, although it seemed to have little effect on results. This was due to his remarkable perceptive faculties, thorough training, and breadth of view, through which he was able to grasp subjects quickly. It was also due to his natural executive capacity. In 1894 his general health began to fail, still further limiting his physical activity. That this in no wise impaired his courage and ability to direct is evident by the progress of the work of the Aqueduct Commissioners after that date and up to the time of his retirement as Chief Engineer, as well as by the important consultations undertaken by him from 1894 to 1901, of which his work as Consulting Engineer to the Metropolitan Water Board of Massachusetts, and as a member of the Comité Technique of the New French Panama Canal Company, can especially be recorded.

Continued failing health impelled his retirement from active duties at the end of 1899, and he died on June 11th, 1903, at his home in Yonkers, New York.

To an attractive and notable personality there was added a remarkable charm of manner which distinguished Mr. Fteley in his relations to all with whom he came in contact. The confidence inspired by his high character, and his kindliness and manifest broad sympathies, caused his subordinates to feel that in him they could always find a friend as well as a respected and acknowledged leader and master, and won the warm regard of every one. His judgments were always based on broad views of affairs, as well as upon intimate knowledge of the essential facts in each question, and they were accordingly abiding. The unlimited patience and generosity which he always exercised toward those who were in any way dependent upon or subordinate to him won their enthusiastic loyalty, and, in consequence, the best that there was in a man was always at his command. This was one of the reasons of Mr. Fteley's remarkable success as an executive, and made it possible for him to accomplish much while the condition of his health was such as to

prohibit physical activity on his part. His last years were years of great suffering, owing to a complication of troubles, but his cheerfulness and fortitude did not fail, and he died as he had always lived, the embodiment of gentleness, patience and courage, having advanced to great achievements in the face of extraordinary difficulties, imposed by ill health, and the cares and anxieties resulting therefrom.

GEORGE SHATTUCK MORISON, Past-President, Am. Soc. C. E.*

Died July 1st, 1903.

George Shattuck Morison was born in New Bedford, Massachusetts, December 19th, 1842.

His father was a Unitarian minister of Scotch-Irish stock which originally settled at Peterboro, New Hampshire, where in later years Mr. George S. Morison made his home, though most of his boyhood was passed at Milton, near Boston, where he developed a faculty for surveying and architecture. He was prepared for college at the Phillips Exeter Academy, and was graduated from Harvard University, with the degree of Bachelor of Arts, in 1863. When at Harvard he was considered the best scholar in mathematics in his class.

Notwithstanding his mathematical inclination, he studied law and was graduated from the Harvard Law School in 1866 with the degree of Bachelor of Laws. He was admitted to the New York Bar the same year. He associated himself with the firm of Evarts, Southmayd and Choate, but, a year later, began his work as a civil engineer, for which he had no special education or training.

As an engineer, he rapidly won distinction and power, and long before his death he had become eminent. He was elected a member of the American Society of Civil Engineers on January 6th, 1875, and in 1895 became its President, the highest strictly professional honor which an engineer can reach in America. He was a Member and Telford Medallist of the Institution of Civil Engineers. He was a Member of the Western Society of Civil Engineers, of which he was for three years a Trustee; a Member of the American Society of Mechanical Engineers; of the American Institute of Mining Engineers; and of the Mexican Society of Engineers and Architects; an Associate Fellow of the American Academy of Arts and Sciences; a Fellow of the American Academy for the Advancement of Science, and a member of various other learned and scientific societies.

He had served on various important engineering boards and commissions, under appointments from the General Government and from State and municipal governments, which will be mentioned later. The most important of these was the Isthmian Canal Commission of 1899 and 1903, the labors of which practically determined the route on which that canal will be built. In brief his reputation as an engineer and engineering counsellor was international.

That a man of academic education, and bred for the law, should have risen so fast and gone so far as an engineer is an extraordinary

*Memoir prepared by E. Gerber, H. G. Prout and C. C. Schneider, Members, Am. Soc. C. E.

and interesting fact. Briefly, he had the sagacity to discover very early in his life that the practice of law was not congenial to his tastes and abilities, and that engineering was. The inexactness of the laws of man could never satisfy him, while the possible exactness of engineering design and construction satisfied his reason; and the possible control of the forces of Nature appealed to his imagination. He was born an engineer, he was not born a lawyer, and he had the courage to change his profession at the outset of his career. The lesson of this interesting life may be suggested by a very brief sketch of his work, and by a few words about his mind and character.

He began his engineering work in October, 1867, on the bridge over the Missouri River at Kansas City, which was being built by Octave Chanute, Past-President, Am. Soc. C. E., and displayed such ability that he was selected to assist in preparing for publication an account of the construction of the Kansas City Bridge, which had become noted for its novelty and difficulties.

From January, 1870, to June, 1871, Mr. Morison was engaged in making examinations of the character and resources of the country through which it was proposed to extend the "Joy roads." In June, 1871, he was called to Detroit to take charge of the Detroit, Eel River and Illinois Railroad, as Chief Engineer. He remained there until April, 1873. By that time, Mr. Chanute having become Chief Engineer of the Erie Railroad, Mr. Morison rejoined him as Resident Engineer of the Eastern Division. The Erie Railroad at that time was chiefly equipped with wooden bridges; the few which were of iron being proportioned for much lighter loadings than those of the new locomotives which it was intended to place upon the line, and Mr. Morison had an ample field for his tastes and talents in reconstructing and strengthening the bridges. He soon became the Principal Assistant Engineer, and when the celebrated Portage Bridge burned down, in 1875, he designed and built the iron structure, which took its place, in six weeks from the date of the conflagration.

He resigned from the Erie Railroad in November, 1875, to become consulting expert on railway properties for Baring Brothers and Company, and was connected with them for ten years. He had shown such individuality that his friends thought that his greatest success would be as a railroad director and manager, and he served as director (representing the Barings) for ten years on the Eastern Railroad of Massachusetts, four years on the St. Louis, Iron Mountain and Southern Railway, eight years on the Maine Central Railroad, and eight years on the Ohio and Mississippi Railroad. the latter directorship extending to 1892.

From 1875 to 1880 he was a member of the firm of Morison,

Field and Company, bridge contractors, from which he retired in order to give all his attention to professional engineering. He at once took a prominent position among consulting engineers by beginning the construction, in rapid succession, of that long list of bridges, over the Missouri and Mississippi and other important rivers, which have made his name so famous.

To bridge the Missouri River in 1880 was still a great undertaking, and had few precedents. His first great bridge was that at Plattsmouth, over the Missouri, and was completed in 1880. It is one of the early bridges in which steel was used to a considerable extent, the two 400-ft. spans being a little more than half steel. The description given by Mr. Morison, in his monograph on the Plattsmouth Bridge, as to how this steel was made, is a bit of interesting history of the early development of the manufacture of structural steel.

Before the Plattsmouth Bridge was opened for traffic, Mr. Morison had let the contracts for the construction of another bridge over the Missouri, at Bismarck, North Dakota. This was completed in October, 1882.

The pneumatic foundations in these two bridges had been sunk under contracts by people more or less familiar with such work. When, however, the bridge at Blair, also over the Missouri, was begun in 1882, Mr. Morison decided to do the pneumatic work himself by day work under experienced foremen, as thereby he could control the work better, use such safety appliances as were not likely to be used by the contractors, and he hoped to achieve some considerable economy. The work at Blair proved so successful that he adopted this plan at nearly all subsequent bridges where pneumatic foundations were required. These came in such succession that he was able to transfer the pneumatic plant from one to the other, and, eventually, a very fine pneumatic plant was developed at a comparatively small cost to any one bridge. The Blair Bridge was opened for traffic in 1883, but the problem there was by no means solved. All Missouri River bridges require more or less shore protection, but at none was the channel so fickle as at Blair, and for several years great sums of money were expended under Mr. Morison's direction for controlling the river, with ultimate complete success.

The old bridge at Omaha being overcrowded with business, and rather light, Mr. Morison was called upon to rebuild it as a double-track structure. The new bridge is essentially new throughout, a few of the old piers only being used for supporting the approach spans. This bridge was practically completed in June, 1888, though it was not used as a double-track structure until October of that year. Before beginning the Omaha Bridge, Mr. Morison had made in-

vestigations for and actually begun work on another Missouri River bridge at Rulo, Nebraska. Work on this, however, was suspended for a time, and the bridge was not opened for traffic until September, 1887.

While these bridges were building, Mr. Morison constructed the following bridges on the Pacific Slope: over the Snake River at Ainsworth, Washington, in 1883; and over Clark's Fork of the Columbia, near Belknap, Montana, in 1884.

Up to this time the superstructure of Mr. Morison's great bridges had been made partly of iron and partly of steel, the floor, intermediate posts and laterals being of iron, the remainder of steel. In his next Missouri River bridge, completed in November, 1888, at Sioux City, iron was used only in members requiring welding; the remainder of the superstructure was of steel, either open-hearth or Bessemer. All Mr. Morison's subsequent bridges of large size were built practically entirely of steel.

In 1887 he formed a partnership with E. L. Corthell, M. Am. Soc. C. E., under the firm name of Morison and Corthell. This partnership continued from May 1st, 1887, to May 1st, 1889. The Sioux City and Nebraska City Bridges, across the Missouri; the Cairo Bridge, across the Ohio; the Jacksonville Bridge, across the St. John's, in Florida; the bridge over the Willamette, at Portland, Oregon; the bridge over the Snake River, at Riparia, Washington; and the Bismarck Water-Works were built, and the plans were prepared for the Merchants' Bridge, over the Mississippi, at St. Louis, under their joint direction. The Cairo Bridge was at that time the longest metal bridge in the world, the metal work being exactly 2 miles long.

Before the Cairo Bridge was completed, Mr. Morison began the great bridge at Memphis, Tennessee. It is his greatest bridge work, is the nearest bridge to the mouth of the Mississippi, when completed had the longest truss span in the United States, and was practically second only to the great structure over the Firth of Forth, although its longest span was a few feet shorter than a span in the Lansdowne Bridge, in India. The pneumatic foundations of the Memphis Bridge were really its controlling features, as they were in unusually deep water (about 40 ft.), and were sunk to the extraordinary depth of 108 ft. below water.

The superstructure presented many novel problems, not the least of which was the character of the material of which it was composed. On a structure of such importance, it was worth while to exercise special care in making the steel, and many conferences were held with the steel manufacturers, and a specification resulted which permitted the use of open-hearth or Bessemer steel. After several hundred tons of the latter had been rolled and it had been

found extremely difficult to meet the requirements, the use of Bessemer steel was abandoned, with the advice and consent of the manufacturers; that already rolled was sold for other uses, and open-hearth steel, either acid or basic, was used exclusively. Basic steel was then in its infancy, and there was much prejudice against it. The specifications finally developed on the Memphis Bridge have been essentially the basis of all modern bridge specifications throughout the United States. The Memphis Bridge was successfully completed and opened for traffic with great ceremony on May 12th, 1892.

While the Memphis Bridge was being built, Mr. Morison built a bridge over the Mississippi at Winona, Minnesota (1891), and, at Burlington, Iowa, he replaced the original single-track bridge by a double-track structure (1891). These were followed by a bridge over the Missouri at Leavenworth, Kansas, one over the same river at Bellefontaine Bluffs, Missouri (1892-93), and one over the Mississippi at Alton, Illinois (1892-93). He rebuilt the bridge across the Missouri at Atchison, Missouri (1898). This completes the list of his great bridges, of which ten crossed the Missouri, five the Mississippi, and one the Ohio. In addition to these large bridges, he built many smaller bridge and viaduct structures in every part of the United States. In the early Nineties he made an examination and report on a bridge over the Detroit River, at Detroit, and was again engaged on this problem at the time of his death.

In 1889 he made an exhaustive report on the Atchison, Topeka and Santa Fé Railway, and in the next few years reported on several other railroad projects of lesser magnitude.

He competed with others in prize designs for two bridges near Washington, and was awarded a prize in each case.

Mr. Morison served on the following Engineering Boards: The Board of Engineers upon the New York and New Jersey Bridge (1894); the Board to locate a deep-water harbor in Southern California (1897); a Board to report on the betterment of the water front of New York City (1895-97); a Board to advise the State Engineer and Surveyor of the State of New York in regard to the plans and estimates for the so-called Barge Canal (1900-01); the Isthmian Canal Commission (1899-1903); a Commission of engineers to report on the plans of the Manhattan Bridge across the East River (1903). He was Chairman of this Commission, and signed its report on June 29th, 1903, only two days before his death, and this was probably the last document signed by him.

Mr. Morison's part in the studies and the findings of the Isthmian Canal Commission must necessarily remain largely unknown to everyone but the members of the Commission; but enough is known to justify the statement that the nation and the world owes him a

good deal. When he entered upon his work with the Commission he had not allowed himself to reach an opinion as to the best route for the canal. He threw himself into the study of the situation with the energy, the determination to know the truth and all the truth, and with the power of investigation and analysis which characterized all his work. He went to Paris and ransacked the archives of the Panama Canal Company; he exhausted the sources of actual information in America, and he went (with other members of the Commission) to Nicaragua and the Isthmus, and examined the ground. There were those who said that he sought the glory of identifying his name with some route other than the two which had come to be accepted as the only practicable ones. This would not have been an ignoble ambition; but his passion for thoroughness is a sufficient explanation of his reluctance to make up his mind. When he decided in favor of the Panama Route he had canvassed the matter so completely that no doubt remained, and he could speak with the conviction and power of knowledge. Apart from his work with the Commission, he delivered a number of addresses before general audiences, which had much weight in making the choice of the Panama Route acceptable, or even possible.

Such is briefly his more important work. It is not necessary to state that such work required a master mind, and when it is considered that Mr. Morison had no special technical training in engineering, but entered the field when he was nearly 25 years of age, it is indeed marvelous. Nature endowed him with a strong intellect and a strong will, and he made the most of them. The whole grand success may be summed up in the word "work." He had no influential friends to help him, whom he did not make himself by his indomitable energy and proven ability. He studied his work carefully and thoroughly, and the minutest detail was not too small to be worked out with the greatest consideration before it was executed. One of his rules was, that if he had five minutes in which to do a thing, he would take three, if necessary, to think it out, and do it in the other two.

In his work he was original, and not merely an imitator or developer of existing ideas. He sought to make the best possible solution of a problem, and not necessarily a solution which had been shown to be a success under similar circumstances. He sought and had a reason for everything, and had the courage to act according to his reason. He did not, however, carry his originality to extremes. Every previous example bearing on a case was carefully studied, and if he found that some existing idea suited his purpose better than any other, he did not hesitate to make use of it and properly gave credit where it was due.

Nor did he fail to consider the commercial practicability of his

designs. His work was no doubt of the very highest order of his time, but he did not make it of such extreme character that it could not be practically attained. It was always a little better than had been done before, but never out of reach, and thus he led in the development of bridge building, the better standards of to-day being about up to his requirements of a decade ago.

While Mr. Morison always studied out and knew every detail of his work himself, he was careful to surround himself with a competent, faithful and conscientious staff. An indefatigable seeker after truth and the best obtainable, himself, he expected his staff to be no less energetic, accurate and conscientious in their work than he, and an indolent or slovenly worker did not remain long in his service.

After leaving Detroit, in 1873, Mr. Morison resided in New York City for fourteen years, then in Chicago until 1896, when he returned to New York. He was a great traveler, his work calling him to all parts of the United States, the Isthmus, and Europe. His business travels were supplemented by a trip around the world, as well as smaller ones to Europe and to our Southern neighbors. He was an accurate and minute observer, and this, together with his studious habits and early education, gave him a wonderful and very extended fund of knowledge, which made him an entertaining conversationalist and a scholarly, concise writer.

Considering Mr. Morison's great intellectual activity, and the wide range of his interests, and considering that his whole life, from childhood to his death, was surrounded by an atmosphere of scholarship and letters, the printed record that he has left behind him seems surprisingly small. But the practicing engineer seldom has time to write much for publication. Mr. Morison wrote and published some admirable monographs describing the most important of his bridges. He contributed valuable papers and discussions to the published Transactions of the Engineering Societies to which he belonged, and he delivered numerous lectures and addresses before college classes, learned societies and various professional bodies. Some of these latter have been more or less completely preserved in print, and doubtless most of them exist in manuscript, for Mr. Morison was a methodical man. If his literary work, both that which was strictly professional and that which was semi-popular, can be collected and made available, a material service will be done for his profession. But Mr. Morison left behind him one literary work which is worth particular mention, and that is the little volume entitled "The New Epoch as Developed by the Manufacture of Power." This is a re-writing and expansion of an address, made as President of the American Society of Civil Engineers, an oration delivered before the Society of Phi Beta Kappa

at Harvard University, a commencement address at the Rensselaer Polytechnic Institute, and an article printed in the *North American Review*. The book which resulted is highly characteristic of the man. It shows the depth and originality of his thought; it reveals his capacity for high conceptions and powerful generalizations, and expresses his constitutional aversion to many words. In 134 small pages, which may be read in an hour, he has compressed the fruit of years of reading and talking and meditation. The fundamental idea which runs through this little book is that mankind is now entering on a new ethnical period. Mr. Morison sees no reason to accept the idea that with the dawn of civilization the ethnical periods closed, but, on the contrary, he believes that when man learned to manufacture power he entered on a new epoch. This bold and inspiring thought he develops in considering various forms of social organization and activity.

He was for fifteen years a Trustee of the Phillips Exeter Academy, and five years President of the Board, resigning as such in June, 1903. To testify to his belief, as a man of science, in the value of classical study, he began the endowment there of the Morison Professorship of Latin.

He planned and directed, as Chairman of the Building Committee, the construction of Soule Hall—a dormitory—and planned the interior arrangement of Peabody Hall—a second dormitory. A third—Hoyt Hall—was designed and built entirely by Mr. Morison. He shared equally with Professor Wentworth the cost of this building, the amount of money he put into it being considered a part of the endowment of the Morison Professorship of Latin. These three are the most serviceable buildings of the Academy. Mr. Morison was also a member of the Building Committee in charge of the construction of the beautiful Alumni Hall, recently completed. It was designed under his direction by Mr. Casey, an architect, of New York.

Mr. Morison was a member of the Finance Committee throughout the greater part of his trusteeship of fifteen years, and its Chairman many years. Many of the funds were re-invested under his direction, and the Academy received the benefit of his services in many ways not commonly known. He left his mark on every side of the Academy life. He loved the school, and gave much of his thought to its betterment. The Academy had the benefit of his ability, not only in its buildings, but in its funds and every part of its life which needed his aid and encouragement.

Mr. Morison's professional position is secure in the work which he did; his activity in other fields is indicated by the brief record of his service to Phillips Exeter Academy; but the commanding place which he took among the men of his generation is only partly

explained by the mere list of his works, long as it is. To understand his position, we must know something of his personal qualities.

He had a powerful intelligence, which would have distinguished him in any calling, and added to that he had in large measure those special gifts which make a man an engineer in spite of accidents of education. He had contrivance; he had a quick and clear perception of cause and effect in material phenomena; he had a feeling for the laws and forces of Nature. So it was not extraordinary that he should have turned from the law to engineering when he was 25 years of age, or that he should have succeeded greatly as an engineer without what is commonly recognized as an engineering education. These special gifts were evident in his childhood, as were the quick and accurate observation and the strong memory by which the born engineer collects and stores the knowledge which makes many of his acts in after life seem to be what we call intuitive. With a strong mind, Mr. Morison had also a strong will. He followed his purposes, great and small, with a persistence and determination which made him hard to work with, but which secured his ends.

Beneath these attributes, which were evident, were others which were not evident to those who knew him but superficially. With all his strength and self-reliance he was a very modest man. In matters where experience had not taught him that he was strong, he was apt to distrust his own judgment. That is, self-reliance with him was largely a product of reason. He was also a diffident man, and had in great measure that reticence about his own affairs which is characteristic of his race and of the region where he was born and bred. These characteristics should be kept in mind by those of his contemporaries who did not know him closely and who try to sum him up as he appeared to them. Finally, in Mr. Morison's attitude toward his fellow man, he belonged to that school of thought of which Herbert Spencer was the most conspicuous representative in modern times. As a matter of principle, and for the ultimate good of society, he would have made every man help himself to the utmost of his power. And yet he did help many with his counsel and his money when he had satisfied himself that they were worthy.

Although Mr. Morison was in his sixty-first year when he died, he was still growing intellectually, and as he was a man of great physical strength and of frugal and abstemious life, and of undiminished energy and abundant means, we can but feel that had he been spared he would have accomplished some work greater than any which he had yet done.

DANFORTH HURLBUT AINSWORTH, M. Am. Soc. C. E.*

DIED APRIL 24TH, 1904.

Danforth Hurlbut Ainsworth was born at Cape Vincent, New York, on March 8th, 1828, and died in Des Moines, Iowa, on April 24th, 1904. His ancestors were among the 20 000 or 25 000 persons who migrated to New England before the Long Parliament, through dissatisfaction with the laws of England, as administered by Charles I and Archbishop Laud, instead of staying at home to put a working edge on their discontent.

Mr. Ainsworth's immediate ancestors were land owners in Roxbury, Massachusetts, and in Woodstock, Connecticut, and in 1774 moved to Vermont. During both the French and Indian and the Revolutionary Wars they took an active part in the service of their country. Mr. Ainsworth's father moved to Cape Vincent, about 80 years ago, where his active business life was spent in lumbering, potash making, storekeeping, shipbuilding, and trading, mostly, to Montreal and Quebec, until the Papineau Rebellion.

In 1846 Mr. Ainsworth entered Geneva (now Hobart) College, where he was a member of the Sigma Phi Society, graduating in 1850. His first work was as Leveler on the enlargement of the Erie Canal. A change in the politics of the State sent him on the construction of the Syracuse and Binghamton Railroad for a year, from which he went to the Mississippi and Missouri, now a part of the Rock Island System, where, under the late S. B. Reed, M. Am. Soc. C. E., he staked out the first 55 miles west of the Mississippi, though neither he nor Mr. Reed were responsible for that location, which was abandoned long ago.

In the spring of 1854, a change in the political convictions of the State having occurred, Mr. Ainsworth went back to the service of the State of New York as First Assistant Engineer on the Erie Canal. This position required an affidavit to each estimate of work done:

"I and my sworn Assistants have accurately measured the work done since the last estimate: that the present estimate is not in excess, and that to the best of my knowledge and belief, all former estimates are correct."

As the last part of this affidavit was not in accord with his knowledge and belief, it was stricken out before signing the affidavit. Although both the Division and State Engineers upheld this decision not to regard the usual affidavit as entirely *pro forma*, so much annoyance resulted that after some 18 months he returned to the Mississippi and Missouri.

* Memoir prepared by Edward P. North, M. Am. Soc. C. E.

From that time until he gave up professional work, virtually all his time was occupied west of the Mississippi, and most of it west of the Missouri. His power of fitting a location to the ground was probably better developed than that of any of his contemporaries, and no Engineer can ask a fairer monument to his painstaking skill than a comparison between the lines of the Chicago, Burlington and Quincy Railroad, east and west of the Missouri, as they were when opened to traffic.

Nor were his services in connection with the Eastern terminus of the Union Pacific Railroad less skilful, though, through local influence on Congress, that company was forced to adopt a line with higher grades, as set forth in several Congressional documents of 1865.

Mr. Ainsworth frequently wrote for technical and other papers, and notably aided the *Railroad Gazette* in its exposure of the fallacious arguments used by the narrow gauge promoters of thirty-five years ago. His "Recollections of a Civil Engineer" (Newton, Iowa, 1901) gives very full details of his professional life.

He was a militant member of the Protestant Episcopal Church, carrying a sound and healthy religion into every-day life, where it was exhibited in a self-respecting regard for the duties and rights of himself and others, and by a high standard of morality and honesty.

Mr. Ainsworth was elected a Member of the American Society of Civil Engineers on March 3d, 1886.

FREDERICK de FUNIAK, M. Am. Soc. C. E.*

DIED MARCH 29TH, 1905.

Frederick de Funiak, Civil and Mechanical Engineer, was born on August 15th, 1839, in Rome, Italy. His father, Count de Funiak, was a Colonel in the French service.

Frederick de Funiak received his early education in Rome. Later, he went to Vienna, Austria, to the Military Academy, and was graduated in 1857 as Lieutenant of Engineers.

He was sent to Egypt as Assistant Engineer of the first railway built in Egypt, from Alexandria to Cairo. He remained in Egypt until 1859, when he was recalled, and served during the Franco-Italian and Austrian campaigns on the staff of General Sonaz. At the Battle of Solferino he was made Captain. After the Peace of Villa-Franco, he served with Garibaldi. In 1862 Mr. de Funiak came to the United States, which were at that time disunited. Having letters of introduction to General Dick Taylor, of the Confederate Army, he made his way to Memphis, Tennessee, and joined General Taylor's brigade as Captain of Engineers, with headquarters at Meridian, Mississippi. He was with this command throughout the remainder of the war.

After the surrender he went to Memphis and taught mathematics and the languages in several schools and colleges, and also did some engineering work for the city in compiling new maps of Memphis.

In 1866 he was appointed Resident Engineer of the Mississippi River Bridge Levees, and was stationed in Washington County, Mississippi. In 1867 he became Assistant Engineer of the Memphis and Charleston Railroad, and built the Tennessee River Bridge, at Athens. In 1869 he was engaged on the water-works commission, determining a water supply for Memphis. In 1870 he became Chief Engineer of the Mississippi Central Railroad.

In 1871 he was sent to Europe as the joint agent of the Mississippi Central, the Memphis and Charleston and the Virginia-Tennessee Air Line, Railroads. In 1872 he built the Ripley Narrow-Gauge Railroad, the first road of this kind built in America. In 1873 he became Chief Engineer of the main line of the Louisville and Nashville Railroad, and in 1874 he was made Chief Engineer of all roads owned and leased by that system. In 1876 he was appointed Chief Engineer and Superintendent of Machinery, and in 1879 he became General Manager of the entire system, which position he held until 1883. During this time he was Chief Engineer of the Henderson Bridge Company, General Manager of the Nashville and Chattanooga Railroad, and President of the Pen-

* Memoir prepared by James Geddes, Esq., and E. C. Lewis, M. Am. Soc. C. E.

sacola and Atlantic Railroad. He resigned all these positions, on account of his health, and went to Carlsbad. On his return to America he never resumed active work.

In 1865 Colonel de Funiak married Miss Olivia Browning, of Memphis, who, with their four sons, survives him at Louisville, Kentucky, where he died on March 29th, 1905, at the age of 65. He was a genial, attractive man; in his youth a daring, dashing fellow, on the lookout for any adventure; a soldier of fortune, withal a man of letters and learning.

He was elected a Member of the American Society of Civil Engineers on May 7th, 1873.

FREDERICK REGINALD FRENCH, M. Am. Soc. C. E.*

DIED NOVEMBER 20TH, 1904.

Frederick Reginald French, son of Horace and Mary E. Gilleter French, was born in West Lebanon, New Hampshire, on September 25th, 1873. He attended the public schools of West Lebanon and White River Junction, Vermont, and entered the Scientific Department of Dartmouth College in 1889. After two years in college, he was employed on the Concord and Montreal Railway, in northern New Hampshire, and for two years was with the Niagara Falls Paper Company, as assistant on mill construction, and during the last year had responsible charge of the surveying and drawing for the tail-race tunnel connecting the wheel-pit of the Paper Company and the tunnel of the Niagara Falls Power Company.

An incident connected with his early work is noteworthy, as an indication of the ability of the young man. The problem of making detailed drawings and patterns for the cut-stone work, at the intersection of the circular paper mill tunnel with the horse-shoe shaped main tunnel, gave the engineers connected with the work much study and discussion. Mr. French, becoming interested, suggested that he could work out a solution. Believing that the young man had little idea of the difficulty of the problem, he was told to take his time and try it. He produced his drawings, and, after careful examination by his superior, they were accepted and he was sent to the quarry to instruct the stone-cutters in preparing the work. Erection proved the problem to have been correctly solved. For a boy with but two years' college training, and no previous experience in stone-cutting work, this was considered quite remarkable.

Mr. French continued in mill construction, having engineering charge for the Shattuck and Babcock Company, in Wisconsin, until the fall of 1894, when he returned to his studies in the Thayer School of Civil Engineering at Dartmouth.

During the two years in the school, he was employed during all vacations, first with Wise and Watson, of Passaic, New Jersey, on construction at the Dundee Chemical Works, and later, under Professor Robert Fletcher, Consulting Engineer, as Engineer on bridge foundations at White River Junction, Vermont.

Upon graduation from the engineering school, in 1896, he became Chief Engineer for the Niagara Falls Paper Company, and had supervision of plans for, and construction of, extensive buildings and special machinery.

From the latter part of 1898 to 1900, he was engaged in waterworks construction in White River Junction, mill construction at

*Memoir prepared by Arthur W. French, M. Am. Soc. C. E.

Wilder, Vermont, and as joint proprietor of a machine shop and foundry business at Niagara Falls, New York.

Compelled by poor health to seek a milder climate, he traveled through Mexico in 1900, and there became associated with Edgar K. Smoot, M. Am. Soc. C. E., on the harbor improvements at Manzanillo. He remained with Mr. Smoot up to the time of his death, serving first as Assistant Engineer, later as Engineer in charge of construction at Manzanillo, and finally as Consulting Engineer, with headquarters in San Francisco.

He designed much of the special machinery for placing the great stone blocks used in the construction of the breakwater, including a steel derrick of simple and economical design, with a capacity of 50 tons, and special cars for the transportation of the stone.

As an engineer, he was possessed of great industry and perseverance, and had a genius for machine design. A keen imagination, which was ever active, was backed by great good sense and a sure knowledge of mechanical principles. It was his fortune to be connected with large enterprises, and his delight was in the solving of difficult problems. Still, he was ever faithful in the smallest details.

A number of useful inventions were made by Mr. French, several machines connected with paper mill work and an "area cableway," for conveying materials to all points within rectangular areas, being the most important.

His quiet and modest manner, together with the sterling qualities of his character, made firm friends for him among all with whom he came in contact.

His unselfish devotion to duty and his courageous fight against disease during the last years have taught his intimate friends to consider him a hero.

Mr. French was married to Martha O. Hathaway, on February 24th, 1898, and he is survived by his wife and one daughter. He was elected a member of the American Society of Civil Engineers on September 7th, 1904.

EDWARD SHERMAN GOULD, M. Am. Soc. C. E.*

DIED JANUARY 24TH, 1905.

Edward Sherman Gould was born in the City of New York on August 13th, 1837, and died of pneumonia, while stopping for the winter in the same city, on January 24th, 1905.

He came of a long line of distinguished native American ancestry on both sides. His father, of the same name, was the son of Judge James Gould, third or fourth of the name in America, of Litchfield, Connecticut, and Sally McCurdy, daughter of General Uriah Tracy, fifth of the name in America. His mother was the daughter of Cornelius Du Bois, fifth of the name in America, and Sarah Platt, daughter of Robert Ogden, fifth of the name in America.

His early education was under the direction of private tutors. In 1855 he went to Europe with his family, and spent several years in travel and study. In 1856 he made a voyage, "before the mast," from Bordeaux, France, to New York. This sea experience was always greatly prized.

In 1858 he commenced his more serious scientific education, entering the Ecole des Mines de St. Etienne, France, as "élève étranger," on application of the Honorable J. G. Mason, then United States Minister to France. On leaving this school he made journeys through the principal coal and iron districts of Great Britain, Germany and Belgium. From 1862 to 1865 he was Secretary to the Honorable John Bigelow, then Consul-General of the United States at Paris.

In 1865 he returned to the United States, and accepted the position of Engineer to the New York and Schuylkill Coal Company, succeeding Charles Macdonald, M. Am. Soc. C. E. His next position was as Division Engineer on the Buffalo Extension of the Atlantic and Great Western Railway, which he resigned in 1866 to become Engineer and Surveyor of the Bricksburg Land and Improvement Company, owning the tract now known as Lakewood, New Jersey. In this position he succeeded the late Samuel H. Shreve, M. Am. Soc. C. E.

In 1868, while in this employment, he married Arabella Duncan, youngest daughter of the late Dr. Edward Greenleaf Ludlow, an eminent New York physician, and the late Mary Kennedy Lewis, great-granddaughter of Francis Lewis, one of the signers of the Declaration of Independence.

In 1871 he was Locating Engineer for the projected Augusta and Hartwell Railway, Georgia, of which Mr. Charles Seymour

* Memoir prepared by S. L. Cooper, M. Am. Soc. C. E.

was Chief Engineer. From 1873 to 1876 he was Civil Assistant Engineer to the late General Q. A. Gillmore, United States Corps of Engineers, on river and harbor improvements in the South, and in the reconstruction of Forts Sumter and Moultrie, Charleston, South Carolina.

In 1877 he became Assistant Engineer in the Bureau of the Croton Aqueduct, Department of Public Works, City of New York, under the late John C. Campbell, Chief Engineer. While in this position he made investigations for an increased water supply from the Housatonic River, and also from the Bronx and Byram Rivers, and was afterward in charge of the construction of the Kensico Reservoir and the upper section of the 48-in. pipe line in connection with the same. Upon the completion of this work he commenced the preliminary surveys for the new Croton Aqueduct.

From 1884 to 1886 he was Division Engineer of the new Croton Aqueduct, but left this position for the reason that his strict sense of duty and high professional honor rendered him *persona non grata* to the contractors. Those who recall the conditions surrounding this work at this period will understand the reason, most creditable to Mr. Gould, that led to the severance of his connection with this work.

From 1886 to 1890 Mr. Gould was Consulting and Executive Engineer to the Scranton Gas and Water Company. During this connection he built the Oak Run, the Dunnings, and the Meadow Brook Dams and Reservoirs. From 1890 to 1894 he was Consulting and Constructing Engineer for Runkle, Smith and Company, American contractors for the new Havana Water-Works, known as "El Canal de Albear." At the same time he was also Engineer in Charge of the Palatino Reservoir.

From 1895 to the time of the Spanish-American War he was engaged as consulting engineer, largely in Mexico, Central America and South America, including projects for water supply, irrigation and railroads. In 1896 he received the decoration of "El Busto del Libertador," from the Republic of Venezuela for services to that country.

During the war with Spain he was on the staff of General Ludlow, at Tampa, Florida, in a civil capacity, to furnish aid and information in connection with the proposed military expedition to Havana. After the war he returned to Havana, Cuba, and was engaged in wharf and railroad construction for the United States Government. Of late years his work has been chiefly consultation, covering a wide range of engineering. At the time of his death he was engaged in planning an improved water supply for the City of Monterey, Mexico. This work was about to commence under his supervision.

For many years, and at the time of his death, Mr. Gould's home was in Yonkers, New York. He had recently been a member of the Commission appointed by the Mayor to represent that city in the elimination of the grade crossings on the Hudson River Railroad, and rendered valuable services in that connection.

Besides his widow, Mr. Gould left three children, John Warren Du Bois Gould, Jun. Am. Soc. C. E.; Francis Lewis Gould, a student now in college, and Susan Mary Gould. Another son, Edward Ludlow Gould, a young engineer of fine attainments, died on April 16th, 1903.

Mr. Gould was a frequent and valued contributor to the engineering and periodical press, as well as to the *Transactions* of the American Society of Civil Engineers. He was the author of the following works: "The Elements of Water Supply Engineering," a valuable work on the subject; "A Primer of the Calculus," "Practical Hydrostatics and Hydrostatic Formulas," "High Masonry Dams," "The Arithmetic of the Steam Engine," and a series of standard specifications for dams and reservoirs.

Mr. Gould was an accomplished linguist and mathematician. His wide travels, observing nature, and fine appreciative and genial character made him a charming personality, and endeared him to those with whom he was associated. He was also most kind and considerate to young engineers, to whom he contributed gladly and freely from his wide store of knowledge. The writer had the privilege of starting his professional career with Mr. Gould some twenty-six years ago, and through all the succeeding years has had reason to be grateful for this early association. He was, withal, a cultured Christian gentleman, an honor to his name and his profession.

Mr. Gould was elected a Member of the American Society of Civil Engineers on November 4th, 1885.

JACOB ALBERT LATCHA, M. Am. Soc. C. E.

Died November 30th, 1904.

The record of the life of J. Albert Latcha, which closed at the City of Coldwater, Michigan, on November 30th, 1904, is one worthy of note. Born in the State of Pennsylvania, of a Huguenot family which had settled in the United States at an early date and held letters patent, for land from the Government, dated in 1784, it was in his youth that he chose the profession in which afterward he rose to eminence.

Having gained a solid education at the West Branch Academy, he supplemented it by a special course in mathematics and engineering under Professor Parker, of the Naval Academy, at Annapolis. This in turn was followed for years, and in fact throughout his entire active life, by a well-selected and systematic course of reading, and by constant study. Those who were admitted to the privilege of his friendship in later years often were surprised by his mental grasp, his wide information, and his ready apprehension of every subject presented.

Beginning his railway experience in 1865, as Assistant Engineer with the Philadelphia and Reading Railroad Company, in the next year Mr. Latcha was appointed to a similar position on the First Division of the Union Pacific Railroad, in making the original surveys across the continent from Kansas City to San Francisco, *via* the 32d and 35th parallels. On this survey, subsequently, were built both the Southern Pacific Railway and the Atchison Railroad. Mr. Latcha continued in charge to the Coast, and returning in 1868, he entered the service of the Pennsylvania Railroad, continuing therein for about nine years, and building most of the important lines west of Pittsburg.

In 1880, Mr. Latcha began the construction of the New York, Chicago and St. Louis Railway, built in the interests of the Seney Syndicate. The preliminary surveys for this railway were begun in February, 1881, and the road from Buffalo to Chicago, 510 miles, was completed and ready for through passenger trains by the middle of August, 1882; being 480 working days, or 18 months.

In November, 1886, Mr. Latcha was appointed Engineer and Superintendent of Construction of the Duluth, South Shore and Atlantic Railway. Preliminary surveys were begun on November 5th; the line was located from the Marquette Iron and Copper Range to Superior in 90 days; contractors began work in February, 1887, and 225 miles of railroad were built and track laid through unbroken forests in 13 months.

At different periods, during the years mentioned above, Mr. Latcha also held the presidency of the Toledo, Tiffin and Eastern Railroad, the Toledo and State Line Railroad, the Union Bridge Company, the Toledo and Milwaukee Railway, the Michigan and Ohio Railroad, and the Toledo, Marshall and Northern Railroad; besides being Chief Engineer and Superintendent of Construction of several of the same companies. Still other railroads, construction works, and companies might be mentioned in which Mr. Latcha was actively interested.

In all his wide and varied experiences, Mr. Latcha was noted for his singularly close and accurate estimates of the cost of construction work of all kinds. This was observed by those at the head of the departments of finance, as well as by those who worked under his direction. His executive ability was marked, and his mental endowments fitted him from early life to be a leader of men.

Regarding his professional work, Mr. George B. Roberts, late President of the Pennsylvania Railroad, once stated before the Board of Directors in the office of the company, at Philadelphia, that Mr. Latcha had done more work in less time and at less cost than any other engineer who had ever reported to him. Mr. John P. Green, Second Vice-President of the Pennsylvania Railroad, in a personal letter, declared Mr. Latcha to be one of the best equipped men of his profession the company ever had in its service.

Mr. Latcha was elected a Member of the American Society of Civil Engineers on May 7th, 1873, and at the time of his death he was not far behind those longest in such continuous membership.

For several years prior to the burning of the Hotel Windsor, Mr. Latcha and his wife lived at that New York hotel, the city indeed long having been considered their place of residence; and both before and during that time they made trips to Europe, the Pacific Coast, and to other parts of this country.

At various times, especially during his residence in New York, Mr. Latcha was an occasional contributor to the *North American Review* and *The Forum*. He was a firm believer in railroads, as against canals, as affording the best possible means for the transportation of grain and other western products to the Atlantic Coast. In several well-considered articles he forcefully advocated the construction of one or more Government freight lines for this purpose, and showed how the result could be brought about within reasonable limits, and the leadership of the United States in supplying the grain markets of the world thus be secured and maintained. Recent movements in Congress, or by Congressmen, indicate that his ideas ultimately may be carried out.

Not long before the conflagration that destroyed the Hotel Windsor, Mr. Latcha purchased and handsomely remodeled the parental

home of Mrs. Latcha, at Coldwater, Michigan, upon the death of her father and mother, intending to use it chiefly as a summer residence. Within a short time after taking possession of the new home, Mr. Latcha suffered a sudden attack, somewhat paralytic in nature, from which, thereafter, he only partially recovered.

Although remaining an invalid for several years, Mr. Latcha never lost his interest in the affairs of the world, and as lately as the week previous to his death he had been consulted upon great enterprises already started and others just developing. When the offer of Mr. Carnegie came up before the Society, Mr. Latcha, with his usual discernment, committed himself to the side which finally won, and no engineer felt more gratification than he over the result.

Throughout his long and, to a man of his strong and self-reliant nature, particularly trying illness, Mr. Latcha displayed a spirit of marked patience and great self-control. The end had long been foreseen by him. Those who were closely allied with Mr. Latcha during the days of his professional activity declare that they have lost a dear and true friend.

At the funeral service, held at Coldwater, on Sunday afternoon, December 4th, 1904, the Rev. H. P. Collin, pastor of the First Presbyterian Church, and a long-time friend of the family, said of Mr. Latcha:

"The memory of his refined, rich, noble nature must ever abide in the minds of those who knew him in the fellowship of home and friends. The longer and more active part of Mr. Latcha's life was not lived in our community. He was prominent in large and important public interests in the industrial world of our country. His mental grasp of railway possibilities was broad, far-reaching, and reliable in an unusual degree. He was patriotic in his spirit, and many of his fellow citizens besides us who are gathered here to-day gratefully recognize the value of his contributions to our nation's life."

In his married life Mr. Latcha was particularly happy. His wife was a daughter of the late Bleecker Lansing Webb, a pioneer dry-goods merchant and business man of Coldwater, Mich., who himself was a native of New York City.

WILLIAM BESWICK MYERS-BESWICK, M. Am. Soc. C. E.*

Died December 27th, 1904.

William Beswick Myers-Beswick was born at Leeds, England, on July 2d, 1850. He was the son of the late Mr. W. H. N. Myers, of Leeds, and took the surname of Beswick on inheriting the ancient manorial property of Gristhorpe, near Filey, in Yorkshire.

He served his articles with Messrs. Filliter and Rofe, Hydraulic Engineers, of Leeds, and afterward went through the civil engineering course at Berlin University. On his return to England he joined the staff of the late Mr. John Fraser, of Leeds. He continued that gentleman's practice, in conjunction with his brother-in-law, Mr. H. J. Fraser, and after the death of the latter continued in practice by himself, with offices in Westminster and Leeds, and, for the four last years, in partnership with Mr. W. P. Morison and Mr. G. F. Murray, both of whom had co-operated with him as assistants for many years.

His practice comprised the construction of many important branches of the Great Northern Railway, in Yorkshire and Leicestershire and works for the North Eastern and London and North Western Companies, including some heavy tunneling, viaducts, and a great variety of bridge work. On the Pudsey Branch he constructed the Tyersall Bank, the extreme height of which is 110 ft., and which contains about 600 000 cu. yd. of material. He also conducted an extensive parliamentary and general consulting practice.

Mr. Myers-Beswick was a Member of the Institution of Civil Engineers, a Member of the Institution of Mechanical Engineers, and a Fellow of the Geographical Society. He was elected a Member of the American Society of Civil Engineers on January 3d, 1894.

Mr. Myers-Beswick died at Malvern on December 27th, 1904.

* Memoir prepared by W. P. Morison, Esq.

ALONZO J. TULLOCK, M. Am. Soc. C. E.*

DIED JULY 21ST, 1904.

Alonzo J. Tullock, the subject of this sketch, died at his home in Leavenworth, Kansas, on July 21st, 1904, at the age of 50 years. He was born on a farm near Rockford, Illinois, in 1854, of Scotch parentage, his father, George Tullock, having come to this country from Edinburgh, Scotland, in the early Forties. His school education was commenced in the country schools near his birthplace, continued in the high school at Rockford, Illinois, in the University of Illinois at Champaign, and finally completed at the University of Michigan, where he received the degree of Civil Engineer in 1876.

In 1878 he was married to Miss Kitty B. Southwick, of Rockford, Illinois. The union was most happy, and was blessed by the birth of three children, a daughter, Florence L., a son, Hubert S., and a daughter, Lucy M. Tullock. Miss Florence L. Tullock has recently been graduated from Smith College, at Northampton, Massachusetts, and the younger daughter, Lucy M. Tullock, is now attending the public schools in Leavenworth. The son, Hubert S. Tullock, at the time of this writing, is a junior in the engineering department of the University of Michigan, his father's Alma Mater.

Soon after his graduation, Mr. Tullock entered the employment of Fox and Howard, a firm of contractors in Chicago, but in 1879 he entered the bridge building firm of Insley, Shire and Tullock, at Leavenworth, Kansas, and from that time until his death he devoted his time principally to bridge, viaduct and wharf construction. At the time of his death he was, and had been for twenty years, the proprietor of the Missouri Valley Bridge and Iron Works, successor to the original firm.

Mr. Tullock built many important bridges in the territory west of the meridian of Chicago, besides building during the early days, between 1880 and 1890, many Howe trusses and combination wood and iron bridges, styles so much in vogue during that period. In the territory mentioned, Mr. Tullock probably built more bridges than any other company or firm. Among the most important structures built by him were: the substructure of a railway bridge across the Missouri River at Leavenworth, Kansas, completed in 1894; a highway bridge across the Missouri River at Jefferson City, Missouri, completed in 1896; a railway bridge across the Red River on the line of the Kansas City Southern (Port Arthur Route); railway bridges across the Arkansas, Canadian and other large rivers, for the St. Louis Southwestern Railway (Cotton Belt Route), Texas

* Memoir prepared by Alfred Noble, A. A. Robinson and E. H. Connor, Members, Am. Soc. C. E.

and Pacific Railway, and other roads at various points. He built the very difficult pneumatic foundations for the Texas and Pacific Railway's bridge across the Atchafalaya, one of the outlets of the Mississippi River into the Gulf of Mexico. Various other pneumatic and deep foundations for bridges across western rivers, which are most difficult to deal with owing to their very steep water-sheds and their extreme freshets, were built by him. His last important work was the construction of the bridges on the St. Louis, Kansas City and Colorado Railway (Rock Island System), among which was one very high structure over the Osage River. This bridge contained one span 375 ft. in length and one concrete pier with a total height of 109 ft., resting upon a pneumatic foundation. This is probably the highest concrete pier yet constructed.

In 1900, Mr. Tullock, in conference with Alfred Noble, Past-President, Am. Soc. C. E., designed the important wharf built by the Mexican Central Railway, for the Mexican Government, at Tampico. Mr. Tullock was made engineer of this important work, and took complete charge of the same during construction. This wharf was built on 454 piers, a large number of which were on pneumatic foundations, reaching the depth of about 50 ft. below mean tide. A full description of this important work has recently appeared in the technical press.

During a busy and studious life, Mr. Tullock accumulated a large and valuable library in which are many rare books, and especially many volumes relating to the Louisiana Purchase, within which territory was located the city chosen for his home.

Mr. Tullock, by his simplicity and force of character and his excellent judgment, attracted to himself men of character and learning. By his uniform courtesy and fair dealing, he commanded the confidence and esteem of those with whom he was associated, and as a friend, he could be depended upon in any emergency.

Mr. Tullock was elected a Member of the American Society of Civil Engineers on June 6th, 1883.

JOHN MILLER CUNNINGHAM, Assoc. M. Am. Soc. C. E.*

Died August 8th, 1904.

John Miller Cunningham was born at Leavenworth, Kansas, on August 27th, 1865.

He was a student in the Class of 1888 of the Rensselaer Polytechnic Institute, and left after completing a special course in 1887. He worked himself upward from the position of draftsman in several institutions, gaining a wide experience, until, as Engineer for the Missouri Valley Bridge Company and the Toledo Bridge Company he entered the field of contracting, in Spokane, Washington, where he built the City Hall and numerous other structures in 1892.

Before the breaking out of the Spanish-American War, he had been engaged in New Orleans upon various constructions for several years prior to his enlistment. He enlisted in May, 1898, in Company C, 2d U. S. Volunteers (Immunes), of which he was First Lieutenant. While stationed at Santiago, he was detailed to raise the gunboat *Barnacoa*. After he was mustered out he returned to Pittsburg, Pennsylvania, where he immediately took up his profession with his accustomed energy.

He had a peculiar fitness for independent work, and possessed, to an unusual degree, that quality of an Engineer, which enables him to undertake with confidence the solution of intricate or involved problems.

To him, Engineering was his life and ambition, and the solution of problems but the incidents that gave it variety.

As an Engineer, he was daring, yet careful. His confidence and courage constituted him a conservative yet untiring worker.

He was married on June 22d, 1888, to Miss Josephine C. Geis, of Erie, Pennsylvania, who survives him.

While engaged in his profession, and while crossing the Mahoning River in the vicinity of his last field of action, the vehicle was suddenly overturned, and an active life full of promise passed away. His sudden demise was a shock to his friends and a loss to the profession.

Mr. Cunningham was elected an Associate Member of the American Society of Civil Engineers on October 7th, 1903, and was, also, a Member of the Western Society of Engineers of Pennsylvania. He was also a Member of the Monongahela Club, and of the Americus Republican Club, of Pittsburg, Pennsylvania.

---

* Memoir prepared by A. L. Schultz, M. Am. Soc. C. E.

VAN NORMAN McGEE, Assoc. M. Am. Soc. C. E.*

Died September 28th, 1904.

Van Norman McGee was born at Bloomington, Indiana, on October 11th, 1873.

He entered the University of Indiana, taking an engineering course. In 1891 he left the University of Indiana to enter the service of the Pennsylvania Railroad. Up to September, 1893, he was, successively, clerk and timekeeper in the office of the Superintendent, rodman, draftsman and instrumentman in the Division Engineers' Corps of the Pennsylvania Lines West of Pittsburg.

In September, 1893, he entered the Leland Stanford, Jr., University, for a year's work, devoting most of his time to the theory of structures. Upon completion of this year's work, he accepted a position as Assistant Engineer of the Indianapolis Union Railway Company, which position he held until March, 1895. During this period he was engaged upon special track work. He left the Indianapolis Union Railway Company to accept a position with the Pennsylvania Railroad, being in the office of the Chief Engineer of the Vandalia Line, at Terre Haute, Indiana. During his stay there he was engaged as draftsman, mostly upon bridge work, which was his forte.

In July, 1896, he went to Chihuahua, Mexico, as instrumentman on location work. Leaving this work, he again entered the Leland Stanford, Jr., University, and was graduated from there with high honors in 1898. He then entered the service of the Southern Pacific Company as Assistant Engineer on the Sacramento Division, which position he held until March, 1900.

Mr. McGee left the Southern Pacific Company to accept the chair in Railroad Engineering in the Imperial Tien Tsin University, China. He arrived in China during the Boxer uprising, and had many thrilling experiences and one narrow escape. The country was so upset that he never assumed the Professorship in the Tien Tsin University, but took a similar chair in the Imperial Nan Yang College, Shanghai, China.

This work he was engaged upon until September, 1902, when, on account of his health, he resigned and returned to the United States. His return was circuitous, and was taken for the purpose of inspecting as many railways as possible, the Siberian Railway being of particular interest to him.

Upon his arrival in the United States, he accepted a position with the New York Central Railroad, working upon the elimination of grade crossings. His health compelled him to leave, and he next

*Memoir prepared by G. D. Stratton, Assoc. M. Am. Soc. C. E.

went to Chico, California, where he did the designing of structures for the Butte County Railway. His health becoming worse, he was compelled to go to Arizona. While there he did some locating for the Clark railway interests. His health became so bad that in February, 1904, he went to Denver, Colorado, the home of his parents. During this month he was compelled to undergo an operation. As soon as possible after this operation he was removed to Colorado Springs, where he passed away.

Mr. McGee's life study was railroad engineering, his favorite branch being bridge design. While in China he collected many valuable data relative to railroads in Japan, but his health never permitted him to bring out a paper on this subject. He was never married. Mr. McGee was a true man and is greatly mourned by all who could call him friend.

Mr. McGee was elected an Associate Member of the American Society of Civil Engineers on June 6th, 1900.

MACY STANTON POPE, Assoc. M. Am. Soc. C. E.*

Died December 10th, 1904.

Macy Stanton Pope was born in East Machias, Washington County, Maine, on July 26th, 1869, of sturdy New England parentage, his father, James Otis Pope, and his mother, Olive Chase, both being natives of East Machias.

His early training was obtained in the public schools of East Machias, and in Washington Academy, located in his native town, from which he was graduated in 1888.

Brought up in a community chiefly interested in lumber and shipping, he spent much of his time in his father's mill, and in the woods, of which his father owned large tracts. He thus acquired, not only an intimate knowledge of these industries and a deep interest in them, but habits of close observation and independent thought, which marked his later life and work.

In the fall of 1888 he entered the Massachusetts Institute of Technology, and was graduated from the Department of Civil Engineering in 1892.

He then entered the employ of the Associated Factory Mutual Fire Insurance Companies, of Boston, taking part in a series of tests upon cast-iron water pipe and fittings being conducted at Nashua, New Hampshire, by John R. Freeman, M. Am. Soc. C. E.

In the following fall he was called to the Massachusetts Institute of Technology as Assistant to Professor Dwight Porter in the Department of Hydraulics, but resigned his position at the end of the academic year to take up practical work again, and to re-enter the employ of the Factory Mutuals Company.

Here his time was divided between the testing department, in which he made tests of various fire protection and prevention devices; the inspection department, in which he visited mills in different parts of the country, and made plans of and reported upon them; and in the private work of Mr. John R. Freeman, then engaged, in addition to his regular duties as Engineer to the Factory Mutuals Company, in preparing plans for the improvement of the Pennichuck Water-Works at Nashua, New Hampshire, and of the power plant of the Piscataquis Pulp and Paper Company, and in various other kindred projects.

In February, 1898, Mr. Pope obtained leave of absence from his company to devote himself to the lumber interests of his family estate. Later, after making a somewhat extended tour, through the southern and western States, with his mother, he resumed his active connection with the company, in June, 1900.

*Memoir prepared by Leonard Metcalf, M. Am. Soc. C. E.

From this time his work lay principally in the inspection department of the company; first, in routine inspection work, later, in special inspections. His early training and natural traits, combined with his personal experience with the practical affairs of business, stood him in good stead and made him a most valuable man for the department. Clear headed, well balanced, and judicially minded, he was well fitted to do the work which fell to him, and merited the words of commendation of one of his associates, who wrote, after Mr. Pope's death:

"It is the verdict of all that the work done in each of these various fields was well done, and that the results were received by those who used them with the fullest confidence. In every case strong common sense and a clear appreciation of relative values were predominating characteristics."

Mr. Pope was much interested in engineering matters, and was a member of various engineering societies, among them The American Society of Civil Engineers, in which he was elected an Associate Member on May 2d, 1900, The Boston Society of Civil Engineers, The New England Water Works Association, The Society of Arts, and of the Technology and Appalachian Mountain Clubs.

He was devoted to his old home and took a warm and active interest in its affairs, as was shown by his presenting to the town of East Machias, jointly with his brothers, John A. and Warren F. Pope, a new bridge across the East Machias River. This structure, a fine three-span, reinforced concrete masonry arch, was built to replace a dangerous old timber crib bridge, not only as a memorial to the Pope family, which had been prominently identified with the affairs of the town for a century, but as an object lesson to the community.

His Alma Mater and the Washington Academy, of which he was a trustee, also claimed Mr. Pope's interest, and were substantially remembered by bequests in his will.

Last summer Mr. Pope took the opportunity to travel abroad for some months for rest and recreation, but, shortly after his return, serious symptoms developed, and he died of acute Bright's disease at Brookline, Massachusetts, on December 10th, 1904.

Quiet and reserved to the world, but a warm and loyal friend, simple in tastes, with high ideals, a well balanced and indomitable worker, Macy Stanton Pope will long be remembered by his friends as a good example of a fine and virile type of New Englander.

NORMAN SMITH LATHAM, Jun. Am. Soc. C. E.*

DIED NOVEMBER 10TH, 1903.

Norman Smith Latham was born at Eastford, Windham County, Connecticut, on June 5th, 1859. He was graduated as Civil Engineer from Sheffield Scientific School of Yale University in the Class of 1882. Immediately on his graduation he was appointed rodman on the South Pennsylvania Railroad, and served on the surveys through Washington and Green Counties in Pennsylvania, and in northern West Virginia. In the following spring he was made Assistant Engineer on preliminary surveys and final location of the Somerset Division of the same road. He remained on this work until February, 1885, and during this time was employed on the curved double-track archway over the Baltimore and Ohio Railroad and the arch over Coxe's Creek.

During 1885 and 1886 Mr. Latham was employed by the late J. W. Shipman as Bridge Draftsman on the design of truss and suspension bridges. From 1886 to May, 1888, he served as Assistant Engineer with Messrs. Buck and McNulty. During this period he had charge of the construction of pile platforms, racks and crib bulkheads for both terminals of the South Brooklyn Ferry; the location of the iron coal trestle and the construction of its pile and masonry foundation, for the Delaware and Hudson Canal Company at Weehawken, N. J.; the location and construction of column foundations for the Manhattan Elevated Railway Company for the connection of the Second and Third Avenue lines at the Harlem River, in addition to drafting and estimating on plans for bridges.

From the fall of 1888 until the summer of 1890 Mr. Latham was employed as Assistant Engineer on the Brooklyn Union Elevated Railroad, first on estimating and drafting, and subsequently on construction; he had charge of the extension of the Fifth Avenue line from Twenty-fifth to Thirty-sixth Streets, in Brooklyn, and laid the track on this line south of Ninth Street.

From July, 1890, to May, 1894, he was employed on the Broadway Cable road, under Major G. W. McNulty, M. Am. Soc. C. E., and was Division Engineer in Charge of the work below Seventeenth Street, including the building of the power-houses at Houston Street and Front Street. This was one of the first and most difficult pieces of cable road construction in New York City, and involved the moving and changing of the sub-surface pipe system on an extensive scale. During 1895 and 1896 he constructed a short section of underground trolley work for the Love Electric Traction Company, building a double-track line from 186th to 194th Street,

* Memoir prepared by O. F. Nichols, M. Am. Soc. C. E.

with the switches and terminal special work; he built the foundation for and set the boiler, engines and dynamos, and completed all the work essential to the safe and successful operation of this road.

During the latter part of 1896 he served in the United States Engineer Department, in the river and harbor work about New York City, particularly at Mt. Vernon, and at the Diamond Reef in the East River. From March, 1897, to 1899 he was Division Engineer for the Metropolitan Street Railway Company, during which time he had charge of building the Fourth Avenue electric railway line south of Forty-second Street, and the Second Avenue line south of Thirty-fourth Street, and all the switches and terminals in Astor Place, at Ninth, Sixth, Broome and Grand Streets.

In 1899 Mr. Latham became General Superintendent for Messrs. Naughton and Company, in building the Eighth Avenue railway line south of Fiftieth Street to Canal Street and Broadway, and in changing the motive power of the Third Avenue surface railway from cable to underground electric service; and changed over the tracks of the Boulevard line on Broadway between 122d and 126th Streets. This work consisted in moving each track 9 ft. sideways, so that they would be 51 ft. instead of 33 ft. apart, from center to center; the work was accomplished successfully and without accident, and the operation of the cars was maintained from the beginning to the end of the work without shutting off the power, stopping the cars or injuring any of the men.

Mr. Latham has thus been closely identified with steam railway and elevated and street railway construction; he was one of the builders of the first, most difficult and most successful of the cable railways in New York City, and subsequently rebuilt this line for use by the underground trolley and extended this construction over a great portion of New York City.

He was a skilful engineer, of excellent training, sound judgment and untiring industry; he was, besides this, a genial gentleman and a most honorable man in all his dealings with men, whether his employees, his associates or his subordinates; he was supremely conscientious in every performance of duty and in all the relations of life, methodical, painstaking and indefatigable in the practice of his profession. All who came to know him well learned to honor and respect him for his genuine worth.

On November 1st, 1888, Mr. Latham married Linda Howell Hackett, of Brooklyn, New York, who, with a daughter, Elizabeth Bell, survives him.

During 1903 he became identified with United States Government work near Albany, New York, pending the development of a railway project in that district, in which he was interested, and he died in Albany on November 10th of that year.

Mr. Latham was above all a manly man; he was modest and large-hearted, a Christian gentleman, a loyal friend, a devoted husband and father. His associates have lost a faithful and devoted friend, the profession a loyal and promising member, and the world a perfectly honest man.

Mr. Latham became a Junior of the American Society of Civil Engineers on July 3d, 1889.

TRANSACTIONS

OF THE

American Society of Civil Engineers.

INDEX.

VOLUME LIV.

JUNE, 1905.

SUBJECT INDEX, PAGE 546.
AUTHOR INDEX, PAGE 550.

Titles of papers are in quotation marks when given with the author's name.

VOLUME LIV.

SUBJECT INDEX.

AUTHOR INDEX.